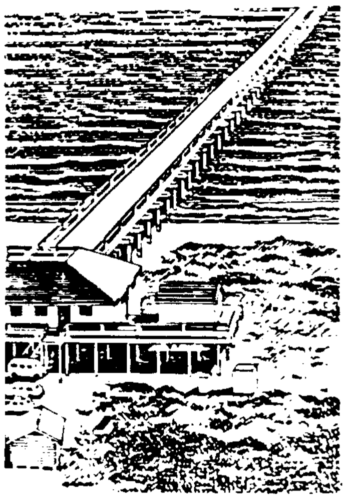

TECHNICAL REPORT CERC-92-12

PHYSICAL MODELING OF SMALL-BOAT HARBORS: DESIGN EXPERIENCE, LESSONS LEARNED, AND MODELING GUIDELINES

by

Robert R. Bottin, Jr.

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

September 1992
Final Report

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Under Work Unit "Shallow Draft Coastal Port Design"

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE
September 1992

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
Physical Modeling of Small-Boat Harbors: Design Experience, Lessons Learned, and Modeling Guidelines

5. FUNDING NUMBERS

6. AUTHOR(S)
Robert R. Bottin, Jr.

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
US Army Engineer Waterways Experiment Station
Coastal Engineering Research Center
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

8. PERFORMING ORGANIZATION REPORT NUMBER
Technical Report CERC-92-12

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
US Army Corps of Engineers
Washington, DC 20314-1000

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

This report summarizes the knowledge and insight gained on small-boat harbor design through years of physical model investigations conducted at the US Army Engineer Waterways Experiment Station. An inventory of physically modeled, small-boat harbor projects has been complied and reviewed. Small-boat harbors modeled and subsequently constructed in the prototype, also have been identified. Site specific performance of these projects has been reviewed to determine if the designs recommended in the model investigations were built as recommended and if they have functioned successfully in the prototype. These reviews and study efforts have resulted in a summary of lessons learned through physical modeling of small-boat harbors and site specific performance in the prototype. Types of small-boat harbors and typical problems, physical modeling in general, and physical modeling guidelines also are included in this report.

14. SUBJECT TERMS
See reverse.

15. NUMBER OF PAGES
170

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		

14. (Concluded).

Harbor classification	Inlet stability	Prototype harbor performance
Harbor design	Modeling guidelines	Small-boat harbors
Harbor shoaling	Physical modeling	Wave action

PREFACE

A research work unit entitled "Shallow Draft Coastal Port Design" was initiated to determine the state of knowledge of small-boat harbor design. Existing design criteria for small craft harbor design is based on "rule-of-thumb" or "down-sized" deep-draft navigation channel design criteria. This research will consolidate existing knowledge on small-boat harbor design and identify research activities needed to fill gaps in existing design criteria. Upon completion, this research will provide comprehensive US Army Corps of Engineers guidance for small-boat harbor design which will be published in an engineer manual.

Overall management of the project is by Headquarters, US Army Corps of Engineers (HQUSACE). Mr. Glenn Drummond, HQUSACE, served as Technical Monitor. This work was carried out at the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), under the general supervision of Dr. James R. Houston and Mr. Charles C. Calhoun, Jr., Director and Assistant Director of CERC, respectively; Mr. Thomas W. Richardson, Chief, Engineering Development Division, CERC; Ms. Joan Pope, Chief, Coastal Structures and Evaluation Branch, CERC; and Mr. Jeff Lillycrop, Principal Investigator, CERC. The work was conducted by and the report prepared by Mr. Robert R. Bottin, Jr., Physical Scientist, CERC, under the direct guidance of Mr. C. E. Chatham, Jr., Chief, Wave Dynamics Division, CERC; and Mr. Dennis G. Markle, Chief, Wave Processes Branch, CERC. This report was typed by Ms. Debbie S. Fulcher, CERC.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4046.856	square metres
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609344	kilometres
pounds (force)	4.4482224	newtons
tons (2,000 lb, force)	8896.444	kilonewtons

PHYSICAL MODELING OF SMALL-BOAT HARBORS: DESIGN EXPERIENCE, LESSONS LEARNED, AND MODELING GUIDELINES

PART I: INTRODUCTION

General

1. A research work unit entitled "Shallow Draft Coastal Port Design" has been initiated to determine the state of knowledge on small-boat harbor design. Existing state-of-the-art design criteria for small-craft harbors is based primarily on "rule-of-thumb" or "down-sized" deep-draft navigation channel design criteria. The study will consolidate existing knowledge on small-boat harbor design and identify research activities needed to fill gaps in existing design criteria. Upon completion, this research will provide comprehensive US Army Corps of Engineers guidance for small-craft harbor design.

2. As part of the work unit, this report summarizes the state of knowledge on design experience gained through physical modeling of small-boat harbors. An inventory of small-boat harbor projects that have been modeled has been compiled and reviewed. This review identifies modifications made to the original project designs as a result of the model investigations. Harbor modifications are generally made to provide adequate or improve existing wave protection, alleviate undesirable current conditions or flood flows, reduce shoaling, or decrease amplification of long period wave energy in the harbor. Small-boat harbors, that have been modeled and subsequently constructed in the prototype, also have been identified in this report. Site specific performance of these projects has been reviewed to determine if the designs recommended in the model investigations were successful in the prototype.

3. These reviews and study efforts have resulted in a summary of lessons learned through physical modeling with respect to small-boat harbor design. As a result of lessons learned through physical modeling of small-boat harbors and site specific performance of various projects in the prototype, physical model usage guidelines have been established. The design engineer may refer to these guidelines to determine where and how physical model investigations are used to solve particular coastal engineering problems. This section will guide planning and design engineers through all stages of

physical modeling. Future research activities, which are required to develop a complete design criteria for small-craft harbor projects, also have been identified.

Types of Small-Boat Harbors and Typical Problems

4. A harbor is an area of water that is protected from wave action to the extent that vessels are provided safe anchorage and mooring, loading, and unloading conditions. Most small-craft harbors are marinas with facilities to moor and service recreational boats or harbors of refuge designed for boats in distress and transient boaters. Small-boat harbors usually have entrance channel depths of 20 ft* or less. A review of over 350 harbor sites with structural components that are under the jurisdiction of the Corps of Engineers has been completed, and a small-boat harbor categorization scheme has been formulated to assist in the development of a comprehensive classification scheme for shallow-draft coastal ports. The review revealed that most harbors are situated within one of three geographical settings which are as follows:

- a. Harbors situated along open coasts. This setting applies to harbors located along the Atlantic, Pacific, and Gulf Coasts, and the coasts of the Great Lakes. With their open exposure to large water bodies with long fetches and distant storms, these harbors are subject to significant wave energy.
- b. Harbors situated in the lee of natural land masses. Numerous harbors have been developed in the lee or shadow of natural land masses, or barrier islands, which provide sheltering from wave energy from some directions of approach. These harbors may be exposed to wave energy from the open ocean from limited directions of approach. They are occasionally situated in large bodies of water where large locally generated waves may cause significant damage during periods of storms.
- c. Harbors situated in small bodies of water. This setting applies to harbors located in lagoons, small lakes, bays, channels, canals, and rivers. They are generally not subjected to large waves, but may experience damage due to locally generated waves associated with storm conditions.

5. Regardless of the geographical settings in which small-craft harbors have been developed, most harbors can be categorized by physical characteristics relative to their geometry and position along the various shorelines.

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* A table of factors for converting non-SI units of measurements to SI (metric) units is presented on page 3.

Classification of harbors and typical engineering problems based on the physical characteristics are discussed in the following subparagraphs.

a. Harbors constructed seaward of the shoreline and protected by breakwaters. Most small-craft harbors fit into this category. Some are built along a straight shoreline and protected entirely by breakwaters, while others are constructed in coves or irregularities in the shoreline, thus utilizing natural protection for waves from various directions and minimizing breakwater lengths. Harbors constructed seaward of the shoreline generally require minimal dredging since their entrances and basins are in relatively deep water. These greater depths, however, require more material for the construction of protective breakwaters. Generally, when breakwaters enclosing a harbor extend to and terminate in relatively deep water, shoaling in the entrance is minimized and maintenance dredging requirements are reduced or eliminated. However, in instances where large volumes of sediment are moving alongshore, material moves around the structures and deposits in the entrance channel, thus, requiring routine maintenance dredging. In some cases, structures extending into deep water may also intercept the movement of sediment. This may prevent natural bypassing and result in accretion on the updrift side and erosion on the down coast side of the harbor entrance. Harbors built seaward of the shoreline often are subjected to large waves in the entrance, which may result in navigation difficulties. Waves propagating directly through the entrance or diffracting through the entrance into the mooring areas may result in undesirable berthing conditions. Wave energy transmitted through breakwaters and wave overtopping of structures may result in adverse conditions in anchorage or mooring areas.

b. Harbors constructed seaward of the shoreline and protected by breakwaters with inner basins built inland through the shoreline. Many harbors have been developed with structures seaward of the shoreline and protected by breakwaters with the addition of inner basins through the shoreline. The inner basins are dredged through the shoreline or are natural irregularities in the shoreline. The inner basins are normally used for small-boat mooring, since they are relatively calm. Harbors of this type may experience the same engineering problems as discussed in paragraph 5a. Also, in some instances, waves propagate into the inner basins and cause undesirable conditions which can damage vessels and facilities. Harbor oscillation problems might occur in inner basins if the frequencies of the modes of oscillation of the basins are similar to the period of incoming wave energy. These conditions may result in amplification of wave heights in antinodal areas (normally in the corners of the basins) and strong horizontal currents in nodal regions, which could cause damage to small-craft and harbor facilities.

c. Harbors constructed inland with an entrance through the shoreline. Harbors of this type are located along numerous coastlines. These harbors normally require more dredging than other harbor types; however, in many instances a channel may be dredged through the shoreline to an existing lake, embayment, lagoon, etc. and result in minimal dredging. Since the harbor is located inland, it is normally more sheltered from storm wave activity and thus requires smaller breakwaters, constructed in shallower water. Since the breakwaters terminate in relatively shallow water (in the breaker zone), problems with breaking waves in the entrance are sometimes experienced as well as strong cross currents which can create hazardous navigation conditions for small craft. Shoaling of the entrance in shallow water is more likely than in deep water due to material being transported in the breaker zone. Other problems that may be experienced with harbors of this type are direct wave penetration into the inner basins and long-period harbor oscillations which may result in facility and/or small-boat damages. On the Great Lakes, seiching may occur and create hazardous current conditions in the harbor entrance.

d. Harbors of the classes in paragraphs 5a, 5b, and 5c with rivers entering into them. Numerous harbors constructed along the coasts as classified by 5a, b, and c have rivers emptying into them. This may present additional engineering problems. River flood flows may cause significant damages to harbor facilities and small-craft vessels. Also, sediment carried downstream by river currents can result in substantial deposits in the harbor and could require maintenance dredging. In some cases, rivers empty adjacent to harbor entrances or in the nearby vicinity. Flood flows may swell over the river banks and cause damages in adjacent harbors. These rivers may also deposit sediments along the coast in the vicinity of the harbor entrance. This material, moved by tides and wave action, could result in entrance shoaling subsequently requiring increased dredging.

e. Harbors constructed inside river mouths. Many small-boat harbors have been built inside river mouths along the various coastlines. These harbors normally require minimal initial dredging. Small boats usually are sheltered from most large waves, and similar to harbors built inland, relatively short breakwater lengths constructed in shallow water are required to provide entrance wave protection. Problems with wave breaking in the entrance, cross-currents, and wave-induced shoaling may occur similar to that of inland harbors. Flood flows also may result in damages in the harbor, and sediment moving downstream could result in shoaling deposits in the lower reaches of the river, thus requiring dredging. In some areas, the passage of ice downstream may cause harbor damages or may result in flooding due to ice jams. River flows opposing incoming waves in the entrance can result in peaked wave crests and treacherous currents and cause navigation difficulties in river mouths. In some cases waves propagate directly up the axis of the river and cause damages to small boats and harbor facilities during storm activity. Other areas have experienced problems with harbor oscillations or surging of basins built adjacent to the river bank which results in damages in the harbor.

f. Harbors constructed in inlets. Numerous harbors of the various classifications have been constructed in inlets along the ocean coasts. These harbors are normally protected from heavy wave action, however, significant problems may result in the inlet entrance. In unstabilized inlet openings, shoaling normally occurs and the entrance meanders due to longshore sediment transport, wave action, and tidal activity. This causes hazardous navigation conditions. To stabilize the inlet opening, dredging and/or the construction of jetties are required to provide protection to the entrance. In some cases, jetties must be long enough to extend beyond the ebb tidal delta. Again shoaling problems and adverse wave and current conditions may be encountered in the jettied entrance which could create hazards to navigation. Tidal exchange between the ocean and embayment may create high-velocity flood and ebb currents through the entrance, and sediment moving alongshore due to wave action may be influenced by these tidal currents and create an unstable meandering entrance condition. Low-crested weirs are installed in some cases to allow sediment to migrate over the jetty. In the lee of the weir a deposition basin is dredged where the sediment falls out and does not come under the influence of tidal currents. Periodic dredging of deposition basins is required to keep them effective or sand bypassing plants may be utilized.

6. In summary, problems normally occur in small-boat harbors in the entrance, in the outer harbor, and/or the inner harbor. Problems in the entrance include shoaling, excessive wave activity, and adverse current conditions. In the outer harbor, anchorage and mooring areas normally encounter problems with excessive wave action that penetrates through the entrance or wave energy that is transmitted through and/or overtops the breakwaters. Inner harbor problems normally result due to wave energy penetrating through the entrance and outer harbor into the basins or due to oscillations in the basins that are forced by longer-period wave energy. Where rivers enter various harbor sites or where harbors are built in river mouths, additional problems are experienced due to river currents, flood flows, sediment movement, and in some cases, ice moving downstream. Harbors built within embayments in tidal inlets may experience the same problems as stated above, however, an additional problem may be navigating the inlet entrance where adverse wave conditions, wave-induced or tidal currents, and/or shoaling may exist.

Physical Modeling as a Design Tool

7. Design of small-craft harbors is very difficult due to the complexity of wave action phenomena and the complicated geometry of most harbors. To provide adequate protection from wave action the design engineer faces the following problems.

_a_. Location of the harbor to ensure that maximum wave protection from wave action is obtained.

_b_. Determination of the location, alignment, height, length, and type of breakwaters required to provide adequate wave protection, and/or jetties required to maintain entrance channels.

_c_. Determination of the best location, orientation, and dimensions of navigation openings to provide vessels safe and easy passage into and out of the harbor without impairing the wave protection characteristics of the harbor works.

_d_. The positioning of spending beaches and other forms of wave absorbers inside the harbor area.

When compounded with problems caused by nearby or adjacent rivers, and/or shoaling problems resulting from littoral transport, and/or harbor oscillation problems relative to long-period wave energy, the designer encounters difficulty in obtaining adequate answers strictly by analytical means. Thus, the hydraulic scale model is commonly used as a design tool to aid in the planning of harbor development and in the design and layout of breakwaters, jetties, groins, absorbers, etc to obtain optimum harbor protection and verify suitable project performance.

8. Hydraulic model studies of small-boat harbors generally are conducted to study the following:

_a_. Determine the most economical breakwater and/or jetty configurations that will provide adequate wave protection and navigation channel control for small craft in the harbor.

_b_. Quantify wave heights in the harbor.

_c_. Alleviate undesirable wave and current conditions in the harbor entrance.

_d_. Study proposals to provide for harbor circulation and/or flushing.

_e_. Provide qualitative information on the effects of structures on the littoral processes.

_f_. Study flood and ice flow conditions.

_g_. Study shoaling conditions at the harbor entrance.

_h_. Study river flow and sediment movement in rivers that may enter in or adjacent to the harbor.

_i_. Study long-period oscillations in the harbor.

_j_. Study tidal currents or seiche-generated currents in the harbor.

_k_. Stabilize inlet entrances.

_l_. Develop remedial plans for alleviation of undesirable conditions as found necessary.

m. Determine if design modifications can be made which could significantly reduce construction costs and still provide adequate harbor protection.

9. To ensure accurate reproduction of short-period wave and current patterns (i.e. simultaneous reproduction of both wave refraction and wave diffraction), geometrically undistorted models (i.e., both the vertical and horizontal scales are the same) are necessary for the study of small-craft harbors. After selection of the linear scale, hydraulic models are designed and operated in accordance with Froude's model law (Stevens et al. 1942). Scale relations commonly used for design and operation of undistorted physical models are shown. A scale of 1:100 (model to prototype) is used in the tabulation for illustrative purposes.

Characteristic	Dimension*	Scale Relations
Length	L	L_r = 1:100
Area	L^2	$A_r = L_r^2$ = 1:10,000
Volume	L^3	$\forall_r = L_r^3$ = 1:1,000,000
Time	T	$T_r = L_r^{1/2}$ 1:10
Velocity	L/T	$V_r = L_r^{1/2}$ = 1:10
Roughness (Manning's coefficient, n)	$L^{1/6}$	$n_r = L_r^{1/6}$ = 1:2,154
Discharge	L^3/T	$Q_r = L_r^{5/2}$ = 1:100,000
Force (Fresh water)	F	$F_r = L_r^3 \gamma_r$ = 1:1,000,000
Force (Salt water)	F	$F_r = L_r^3 \gamma_r$ = 1:1,025,641

* Dimensions are in terms of force, length, and time.

10. Small-scale models must be constructed very accurately to reproduce conditions in the prototype. The reproduction of the most current underwater contours is critical for the correct transformation of waves as they approach the harbor. Shoreline details and irregularities also are important to simulate correct diffraction, runup, and reflective characteristics. The model bed generally is constructed as smooth as possible to minimize viscous scale effects, except in areas such as riverbeds where roughness must be added to correctly simulate flow conditions. Adjustments in model armor and underlayer stone sizes are made, based on previous research and experience, to reproduce correct prototype transmission and reflection characteristics of various structures.

11. Upon completion of model construction, it is essential that representative test conditions be selected. Wave characteristics, direction

of approach, and frequency of occurrence are very important. Refraction analyses are normally required to transform deepwater waves to shallow-water values at the location of the wave generator in the model. From this point, model bathymetry will correctly transform the wave characteristics to the harbor area. Still-water levels (swl's) are also important test conditions. Normally more wave energy reaches a harbor with the higher swl's, but lower swl's may result in more seaward movement of longshore sediment (i.e. around the head of a jetty), since the breaker zone is farther offshore. Dominant movement of wave-induced currents and sediment transport patterns are required for verification of the model as well as river discharge and/or tidal flow information, if applicable to the study.

12. Three-dimensional wave action model studies have been conducted at the US Army Engineer Waterways Experiment Station (WES) since the 1940's. Waves were generated by monochromatic (constant wave period and height) wave generators, and wave data collected with probes that were connected to oscillograph recorders. The output from the oscillograph recorders was analyzed by hand to determine wave heights at various locations in the model. In the 1970's, automated data acquisition and control systems (ADACS) were developed at WES. In these systems, ADACS recorded onto magnetic tapes the electrical output of parallel-rod, resistance-type wave gages that measured the change in water surface elevation with respect to time. The magnetic tape output then was analyzed to obtain accurate wave height data. Currently, ADACS are controlled by MICROVAX computers and capacitance-type wave gages have been developed to obtain wave height information. Capacitance gages are easier to calibrate and more stable than resistance gages since calibration coefficients fluctuate very little with change in temperature. In the 1980's, electrohydraulic, unidirectional spectral (varying wave periods and heights) wave generators were developed at WES. Waves generated from these machines are closer to nature than the monochromatic waves previously used. A directional spectral wave generator capable of reproducing "real world" waves also has been designed and constructed and currently is being used in both research and site specific projects. These generators are controlled and operated with computers.

13. The reproduction of river discharges and steady-state tidal flows often is required in wave action model studies. These flows generally are reproduced using circulation systems (i.e., for a river discharge water is normally withdrawn from the perimeter of the model pit area and discharged in

a stilling basin which empties into the upper reaches of the river and flows downstream in the model). The rate of flow in the past was determined through the use of Van Leer Weirs, manometers, etc. Currently, discharges are measured with magnetic flow tubes and transmitters installed in the pipeline which provide digital readouts. Steady-state flow velocities in hydraulic models may be obtained by timing the progress of a weighted float over known distances on the model floor. Electronic current meters, however, are more commonly used. Wave-induced current velocities are measured by timing the progress of a dye tracer over a known distance on the model floor. Laser doppler velocity meters that are responsive enough to obtain the dynamic velocities in the wave field have been purchased, and operational procedures are in developmental stages.

14. Reproducing the movement of sediment in small-scale coastal model investigations is very difficult. Ideally, quantitative, movable-bed models best determine the effectiveness of various project plans with regard to the erosion and accretion of sediment. This type investigation, however, is difficult and expensive to conduct and entails extensive computations and prototype data. In view of these complexities and due to time and funding constraints, most models are molded in cement mortar (fixed-bed) and a tracer material is selected to qualitatively determine the degree of movement and deposition of sediments in the study area. In past investigations, tracer was chosen in accordance with the scaling relations of (Noda 1972), which indicates a relation or model law among the four basic scale ratios (i.e. the horizontal scale, the vertical scale, the sediment size ratio, and the relative specific weight ratio). These relations were determined experimentally using a wide range of wave conditions and bottom materials, and they are valid mainly for the breaker zone. This procedure was initiated in the mid-1970's, and has been successful in reproducing aspects of prototype sediment movement as evidenced by the performance of completed projects that have been studied. Currently, research is being performed to better understand aspects of sediment movement and improve methods to model it, and scaling relations have been developed for midscale, two-dimensional model tests (Hughes and Fowler 1990).

15. In summary, the small-scale model has played an increasing role as a design tool for coastal structures in the United States since the 1940's. Model techniques and procedures have been developed, instrumentation has improved, and simulation of more complicated phenomena has become possible through experience and basic and applied research. Engineering experience and

the use of analytical methods are still important factors in the design of coastal projects. For many complex coastal engineering problems, particularly those concerning short-period wave effects, the best approach for determining the optimum plan of improvement with respect to wave action, navigation, sediment movement, economics, etc. is through the use of a small-scale physical model. A close alliance between the design engineer and the laboratory engineer should be maintained.

PART II: PHYSICALLY MODELED HARBORS

Inventory

16. Physical model testing of 55 small-boat harbor projects in the United States and/or its territories has been conducted at WES since the 1940's. These model studies have been conducted for 8 sites in Hawaii, American Samoa, Guam, and Alaska; 19 locations on the Pacific Coast, 1 project in Puerto Rico, 1 in the Bahamas, 9 sites on the Atlantic Coast, and 17 locations on the Great Lakes. The locations of these sites are shown in Figure 1. Numbers on the figure correspond to the locations shown in the following tabulation.

Number	Location
1	Agana Small-Boat Harbor, Territory of Guam
2	Taú Harbor, Island of Taú, American Samoa
3	Waianae Small-Boat Harbor, Oahu, Hawaii
4	Kewalo Basin, Oahu, Hawaii
5	Magic Island Complex, Oahu, Hawaii
6	Kawaihae Harbor, Hawaii
7	Laupahoehoe Point, Hawaii
8	St. Paul Harbor, St. Paul Island, Alaska
9	Siuslaw River, Oregon
10	Port Orford, Oregon
11	Rogue River, Oregon
12	Crescent City Harbor, California
13	Noyo Harbor, California
14	Fisherman's Wharf, San Francisco Bay, California
15	Half Moon Bay Harbor, California
16	Monterey Harbor, California
17	Port San Luis, California
18	Santa Barbara Harbor, California
19	Ventura Harbor, California
20	Port Hueneme, California
21	Marina Del Rey, California
22	Redondo Beach King Harbor, California
23	Fish Harbor, Los Angeles, California

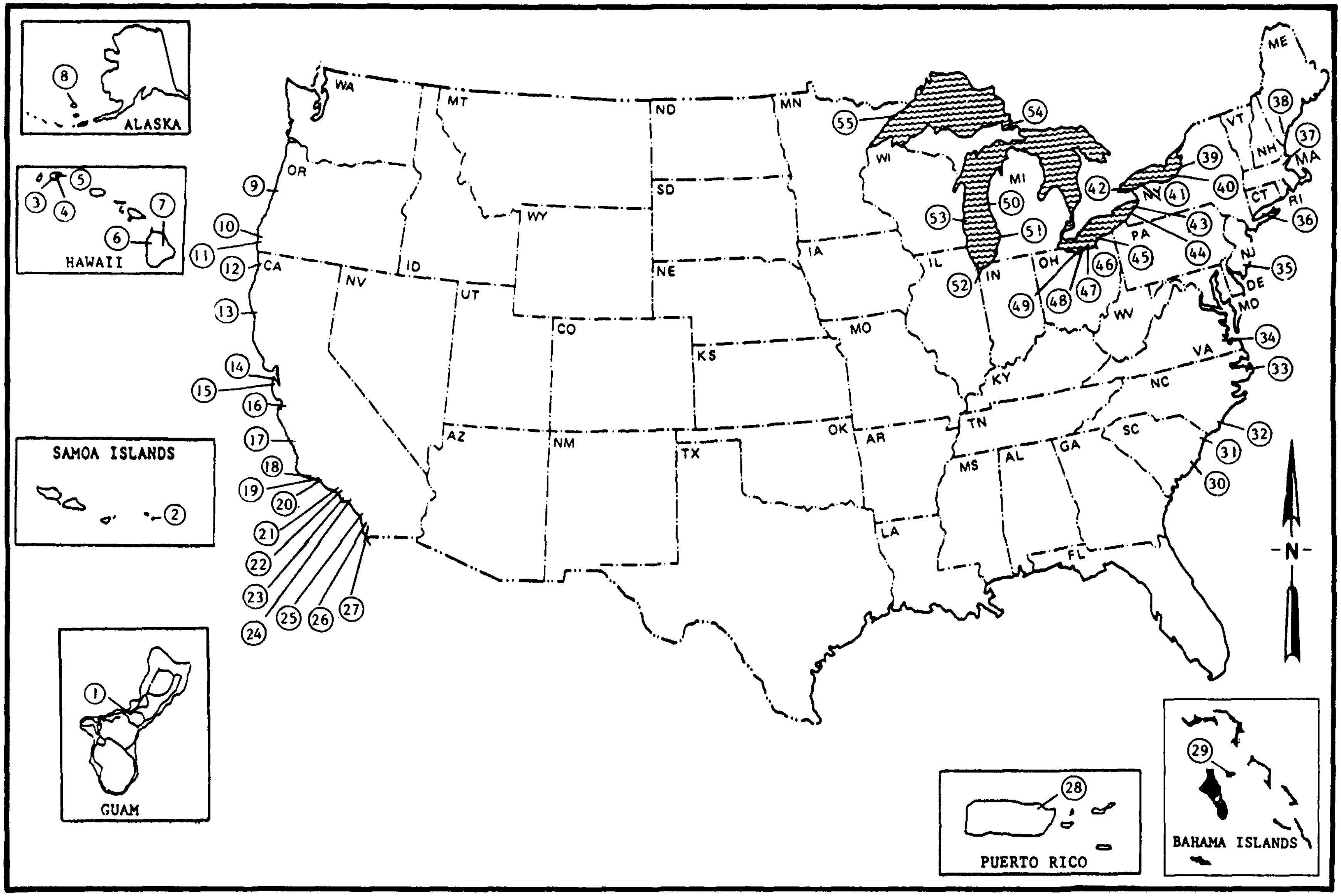

Figure 1. Locations of sites in which physical model investigations have been conducted

24	Bolsa Chica Harbor, California
25	Dana Point Harbor, California
26	Oceanside Harbor, California
27	Mission Bay Harbor, California
28	San Juan Harbor, Puerto Rico
29	Nassau Harbor, Bahamas
30	Murrells Inlet, South Carolina
31	Little River Inlet, South Carolina
32	Masonboro Inlet, North Carolina
33	Oregon Inlet, North Carolina
34	Newport News Harbor, Virginia
35	Barnegat Inlet, New Jersey
36	Shrewsbury Inlet, New York
37	Newburyport Harbor, Massachusetts
38	Wells Harbor, Maine
39	Port Ontario Harbor, New York
40	Oswego Harbor, New York
41	Hamlin Beach Harbor, New York
42	Olcott Harbor, New York
43	Cattaraugus Creek Harbor, New York
44	Barcelona Harbor, New York
45	Conneaut Harbor, Ohio
46	Geneva-on-the-Lake Small-Boat Harbor, Ohio
47	Chagrin River, Ohio
48	Edgewater Marina, Ohio
49	Vermilion Harbor, Ohio
50	Ludington Harbor, Michigan
51	New Buffalo Harbor, Michigan
52	Gary Harbor, Indiana
53	Port Washington Harbor, Wisconsin
54	Little Lake Harbor, Michigan
55	Grand Marais Harbor, Minnesota

Brief Case Histories

17. Physically modeled small-boat harbor investigations conducted at WES are briefly examined in this portion of the report. Harbor design changes

resulting from the model studies are highlighted to assist in defining design experience gained through physical modeling.

Agana Small-Boat Harbor, Territory of Guam

18. Agana Small-Boat Harbor is located on the west coast of the Island of Guam. A shallow natural channel, created by the flow of Agana River over the reef, provides access to two small-boat basins. A very sharp reverse bend in the entrance channel makes navigation difficult. In addition, wave-induced currents in the channel, particularly at the mouth, have compounded the navigation problem. During the winter, high seas and swells with waves up to 12 ft in height prevent passage both in and out of the harbor.

19. Improvements to the existing harbor were required to minimize existing hazardous navigational conditions through the entrance channel and to expand berthing capacity to meet present and future boating needs. A 1:50-scale hydraulic model was constructed and tested to determine the optimum harbor configuration with respect to wave protection, circulation conditions, navigation, and cost (Chatham 1975). Test waves with periods ranging from 8 to 18 sec and heights ranging from 3 to 11 ft were generated from four deep-water directions with swl's of 0.0- and/or +2.4-ft mean lower low water (mllw).

20. Model tests were conducted for existing conditions and 19 test plan configurations. The originally proposed harbor design consisted of new berthing areas, a revetted mole, a 350-ft-long west breakwater and a 175-ft-long east breakwater seaward of the existing harbor. Culvert pipes, 5 ft in diameter, were provided for harbor circulation. In addition, a sewage treatment plant was proposed adjacent to the mole at two different (seaward and shoreward) locations. The proposed plan provided wave protection in the berthing areas, but very confused wave and current patterns existed at the harbor entrance and circulation in the harbor basins was generally poor. As a result of the model investigation, the west and east breakwaters were reoriented and increased in length to 650 ft and 325 ft, respectively. This plan (Figure 2) provided optimal navigation conditions (i.e., crosscurrents were eliminated and wave patterns less confused). It also was determined through the model study that an arrangement of open channels would provide adequate circulation in the berthing areas of the harbor. Optimal channel arrangements were developed for both (seaward and shoreward) locations of the proposed sewage treatment plant.

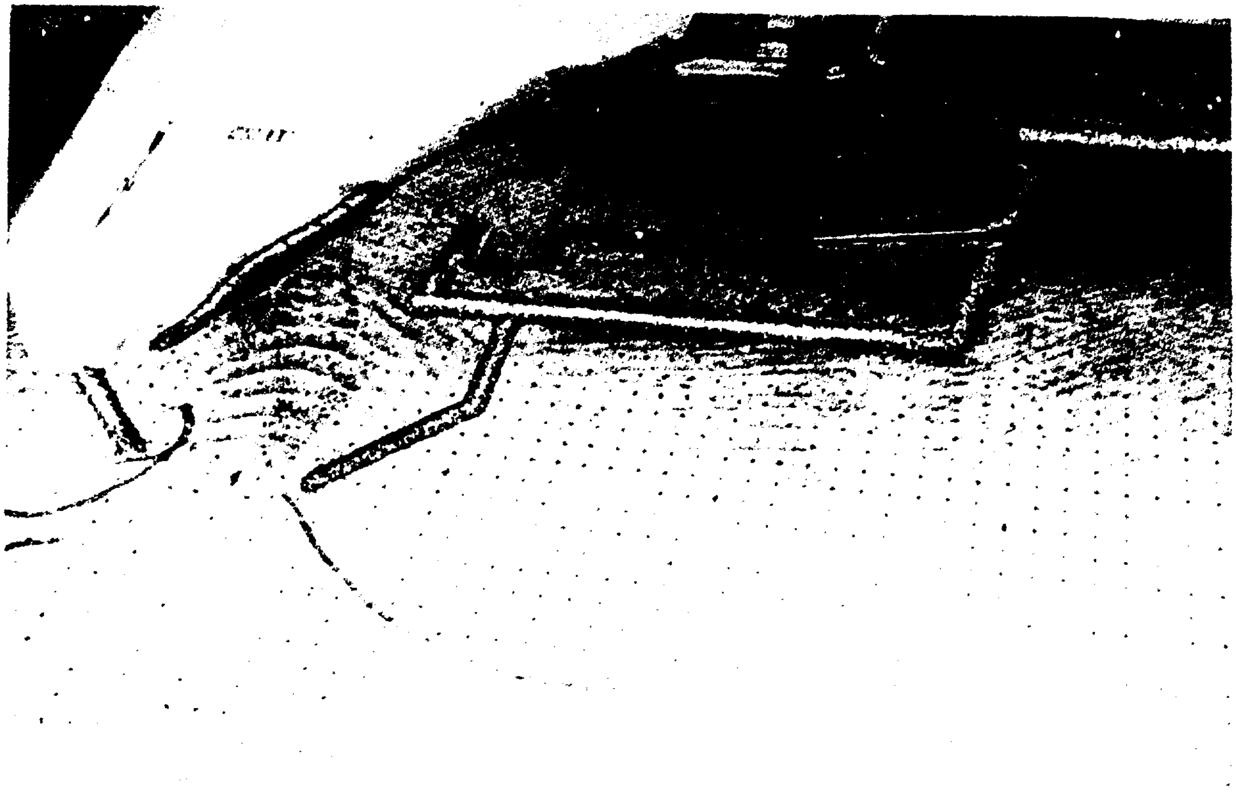

Figure 2. Optimal breakwater configuration for Agana Small-Boat Harbor, Territory of Guam

Taú Harbor, Island of Taú, American Samoa

21. The Island of Taú is located in the American Samoa chain. Transportation of cargo and personnel between islands is accomplished by an inter-island tug and barge network. With barges moored outside the island reefs, loading and unloading of cargo and passengers is accomplished by means of smaller longboats, which travel over the reefs through the "surf zone." Overturning of these longboats has resulted in drownings and loss of cargo. The American Samoan government proposed small-boat harbors on each of its major islands, and the site chosen for Taú was on the western shore of the island.

22. A 1:50-scale hydraulic model was constructed and tested to determine the optimum harbor configuration with respect to wave protection, navigation, beach protection, and cost (Crosby 1974). Test waves with periods ranging from 6 to 18 sec and heights from 4 to 15 ft were generated from three deepwater directions with a +2.8-ft swl mllw.

23. Model tests were conducted for 12 test plan variations of two basic harbor designs. Each harbor configuration had a 350- by 250-ft rectangular basin with a 100-ft-wide entrance channel and a revetted landfill. The first

had an entrance channel connecting the southeast corner of the basin with the ocean, and a breakwater structure and revetted landfill to provide protection. The second had an entrance channel connecting the southwest corner of the basin with the ocean with a groin and a breakwater connected to a revetted landfill providing protection.

24. Tests indicated the first basic harbor configuration ineffective in achieving the desired wave height criteria and revealed cross currents in the entrance channel for all test directions. It also was noted that the revetted landfill was damaged since it was close to the edge of the reef. The second configuration landfill was, therefore, moved 100 ft shoreward, and the severity of overtopping of the revetment was reduced. Increases in the length of the groin resulted in reduced wave heights in the harbor, but significant reduction of wave heights in the mooring area was not obtained. None of the plans tested satisfied the sponsor's 3.0-ft wave height criterion in the berthing area for all test waves. The second harbor configuration (Figure 3) came closest to meeting the criterion but was acceptable only if recognized that there would be periods when the harbor would not be usable. Cross currents and breaking waves were also observed in the entrance for test waves from various directions.

Waianae Small-Boat Harbor, Oahu, Hawaii

25. Waianae is located on the west coast of the island of Oahu, Hawaii, about 30 miles from Honolulu. The Waianae coast is an excellent boating area, and the waters offshore provide some of the best fishing in the Hawaiian Islands. Pokai Bay Boat Harbor served the site, however, it was frequently shoaled in due to littoral material being trapped from the north. In addition, the harbor was severely overcrowded and there was considerable conflict between swimmers and boaters. A new harbor was recommended at the site and subsequently authorized by Congress.

26. Due to physical limitations of the project area, direct exposure to severe wave attack, and the hydrographic factors affecting actual wave conditions, a hydraulic model study was conducted. A 1:50-scale hydraulic model was constructed and tested to aid in the development of a satisfactory harbor configuration and entrance channel location and alignment; determine wave heights at critical areas within the harbor; optimize the length, alignment, and crest el of the breakwaters; and determine wave-induced circulation and shoaling patterns (Bottin, Chatham, and Carver 1976). Test waves with periods

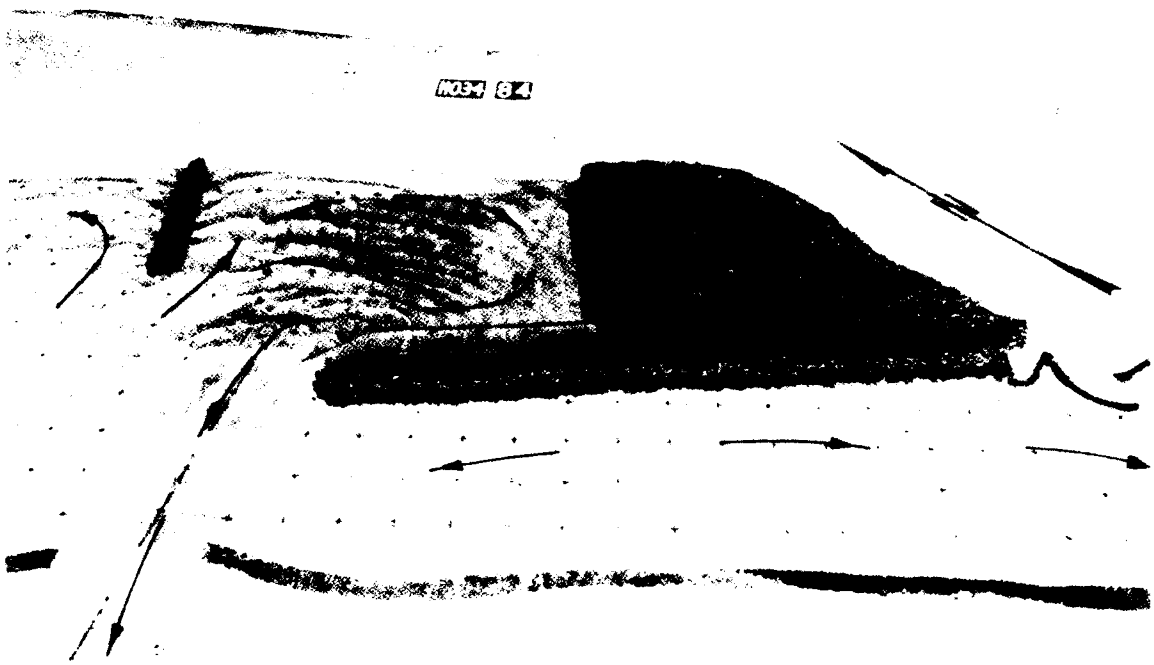

Figure 3. Second basic harbor configuration for Taú Harbor, American Samoa

ranging from 8 to 18 sec and heights from 4 to 12 ft were generated from four deepwater directions with a +3.0-ft swl mllw.

27. Model tests were conducted for the existing site and 12 test plan variations of the basic harbor design. The originally proposed harbor design consisted of a 1,650-ft-long outer breakwater, a 150-ft-long stub breakwater, an entrance channel, turning basin, two revetted fill areas inside the harbor, and a boat launching ramp. Results indicated that the design met the sponsor's established wave height criterion of 2.0 ft in the harbor berthing areas. It was noted that most wave energy reaching the harbor interior was due to diffraction through the entrance rather than overtopping of, or transmission through, the breakwaters.

28. As a result of the model investigation the seaward end of the outer breakwater was reoriented and the entire structure's crest width was reduced from +22 ft to +15 ft. The stub breakwater was increased in length from 150 ft to 250 ft, but its cross section was significantly reduced (from a +15-ft crest height to +8.5 ft; from 3- to 5-ton armor stone to 800- to 1,500-lb armor; and from 15-ft crest width to 12-ft crest). Wave conditions

were similar in the berthing areas, but construction costs were substantially reduced. Wave-induced circulation and shoaling patterns also appeared favorable for the new configuration. A view of the optimum plan is shown in Figure 4.

Figure 4. Optimum harbor configuration for Waianae Small-Boat Harbor, Oahu, Hawaii

<u>Kewalo Basin, Oahu, Hawaii</u>

29. Kewalo Basin is located on the south coast of Oahu, Hawaii, between Honolulu Harbor and Waikiki. The harbor was dredged into the coral reef, and a protecting landfill was formed on the sides of the basin. The entrance channel was exposed to storm-generated waves which resulted in difficult and dangerous navigation for small craft entering the harbor. Specific problems included cross currents in various portions of the channel, peaking and breaking waves in the entrance channel, and undesirable wave action in the basin.

30. A 1:75-scale model was designed and constructed to study wave and current conditions in Kewalo Basin and its entrance channel (Giles 1975). Test waves with periods ranging from 8 to 18 sec and heights from 6 to 14 ft were generated from three deepwater directions with a +2.0-ft swl mllw. Tests were conducted for existing conditions and 13 improvement plans. The initial

improvement plan consisted of a wave absorber along the shelf bordering the entrance channel. It was effective in reducing wave heights in the basin but had little effect on the strong crosscurrent patterns in the entrance.

31. The model investigation revealed that a 500-ft-long jetty extension (with channel wave absorber) would result in reduced wave heights in the basin and eliminate channel eddy currents. Strong crosscurrent magnitudes would also be shifted seaward (out of the entrance channel) and thus provide better navigation conditions. A view of this plan is shown in Figure 5.

Figure 5. Optimum plan for reduction of entrance channel crosscurrents at Kewalo Basin, Oahu, Hawaii. Wave-induced current patterns and magnitudes (feet per second) are also shown

Magic Island Complex, Oahu, Hawaii

32. A large artificial island was proposed along Ala Moana Park in the city of Honolulu, Hawaii, for recreational and associated activities. When completed, the proposed project would provide land areas suitable for resort hotels, recreational parks, and related activities and would also provide several thousand linear feet of additional beach frontage which would greatly

alleviate highly congested conditions presently existing in the Honolulu Beach areas. The characteristics of waves and currents experienced in the area and unknown effects of these phenomena on the proposed construction led to the recommendation of a model investigation.

33. A 1:100-scale hydraulic model was designed and constructed to develop a satisfactory circulation system through the inner lagoon to prevent stagnation; determine the possibility of pollution in the inner lagoon due to pollutants from a drainage canal; and study wave action in the adjacent harbors of Kewalo Basin and Ala Wai Boat Harbor as a result of the construction (Brasfeild and Chatham 1967a). Test waves with periods ranging from 8 to 18 sec and heights ranging from 6 to 18 ft were reproduced from five deepwater directions for swl's of +1.5- and/or +3.5-ft mllw.

34. Model tests were conducted for 17 test plan configurations to develop satisfactory current conditions in the complex. The original plan consisted of a proposed artificial island and a peninsula (Kewalo Peninsula) west of the island. These landfills were positioned to form an inner lagoon in the lee of the island and circulation channels between the island and the existing Ala Wai Peninsula and the proposed Kewalo Peninsula on the east and west sides of the island, respectively.

35. Model tests indicated that construction of the proposed peninsula would aggravate unfavorable wave conditions already existing in Kewalo Basin. Absorbers installed along the channel sides in conjunction with revisions to the entrance channel (indented wave traps) were not as efficient in reducing wave heights in the basin as absorbers alone in the entrance channel. The optimum plan for Kewalo Basin as a result of the Magic Island complex is shown in Figure 6.

36. The installation of the proposed Magic Island and Kewalo Peninsula were not considered to influence wave heights in Ala Wai Boat Harbor based on wave height tests. Model tests were conducted for expansions of the harbor, however. A revetted mole and two additional basins were proposed. Results indicated that undesirable wave conditions may exist in the entrances to the newly formed basins, but can be alleviated by the addition of rubble-mound wave absorbers at critical locations in the entrance channel (Figure 7).

Kawaihae Harbor, Hawaii

37. Kawaihae Harbor is located in Kawaihae Bay on the northwest coast of the Island of Hawaii. A 2,650-ft-long rubble-mound breakwater was located approximately 400 ft seaward of the harbor basin for protection from storm

Figure 6. Optimum remedial plan for Kewalo Basin with the proposed magic Island Complex installed

waves. There was an urgent need to modify the harbor to provide greater maneuvering area for safe navigation of vessels and reduce wave action within the harbor. It was proposed to widen the channel and enlarge the harbor for bulk sugar carriers. The existing small-craft marina near the harbor entrance would be abandoned and a new small-craft marina was proposed in the rear of the harbor.

38. A 1:100-scale hydraulic model investigation was conducted to study wave action in the existing harbor and entrance followed by testing of proposed harbor revisions; develop remedial plans for alleviation of undesirable navigation conditions as well as wave conditions in the harbor basin; and study wave conditions in the proposed small-craft marina (Brasfeild and Chatham 1967b). Test waves were generated for periods ranging from 8 to 18 sec and heights from 8 to 23.5 ft from six deepwater directions with swl's of +2.5- and/or +3.5-ft mllw.

39. Model tests were conducted for existing conditions and 18 variations in design elements of the proposed improvement plans. The original improvement pl.n consisted of a 500-ft-long rubble-mound wave absorber on the shoreline in the vicinity of the harbor entrance. Tests indicated that this

Figure 7. Proposed expansion of Ala Wai Boat Basin with absorbers installed in entrances of new basins

plan offered no significant improvement over existing conditions with regard to wave heights in the harbor. Absorber lengths up to 800 ft in conjunction with a 750-ft-long breakwater extension were required to provide a more favorable wave climate in the harbor than was present with existing conditions. The most satisfactory current conditions in the entrance were achieved, however, with a 750-ft-long breakwater extension and a new 600-ft-long shore-connected breakwater.

40. Tests indicated that wave heights in the proposed small-boat harbor in the rear of the basin, with a proposed 1,075-ft-long offshore structure and a 300-ft-long shore-connected structure, would be within the sponsor's specified 1.5-ft criterion for most conditions, although this value may be exceeded in the small-boat basin entrance. The optimum plan developed during model study is shown in Figure 8.

Laupahoehoe Point, Hawaii

41. Laupahoehoe Point is on a peninsula on the northeast coast of the island of Hawaii about 25 miles north-northwest of Hilo. The existing small-craft launching ramp was unsafe due to wave energy reflecting off the adjacent rocky shoreline and creating unacceptable conditions at the launching ramp. A

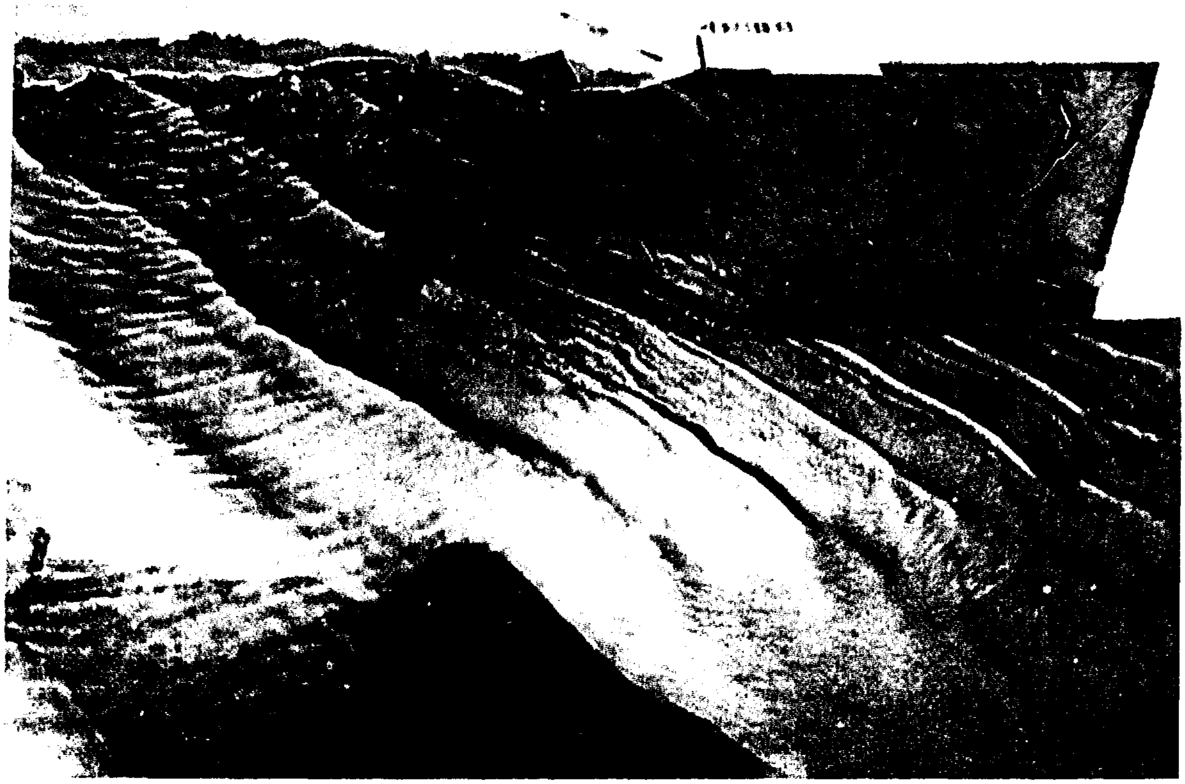

Figure 8. Optimum harbor improvements at Kawaihae Harbor, Hawaii

protected boat-launching ramp was needed that would allow commercial fishermen to take full advantage of the ocean's resources in the immediate area, as well as allow the launching of rescue boats.

42. A 1:52-scale hydraulic model was designed and constructed to determine the optimum length, alignment, crest elevations and stability of proposed structures at the site (Bottin, Markle, and Mize 1987). Test waves (excluding stability test conditions) with periods ranging from 6 to 14 sec and heights ranging from 4 to 20 ft were generated from four deepwater directions with a +2.4-ft swl mllw.

43. Model tests were conducted for 12 test plans. The original improvement plan consisted of an entrance channel, a turning basin, a 200-ft-long rubble-mound breakwater (seaward end having a rib cap and armored with dolosse), and a 60-ft-long rubble wave absorber installed adjacent to the shoreline. This plan resulted in wave heights within the established 2-ft wave height criterion (for deepwater waves of 6 ft or less) in the turning basin about 69 percent of the time (based on hindcast data). Tests without the wave absorber revealed that the breakwater alone was ineffective in reducing wave heights to the desired criteria. Also the length and alignment of

the original absorber was found to be optimum since additional wave absorber length had little effect on further reducing wave heights in the area. Tests also indicated that sealing of the breakwater had little effect on wave heights in the new basin. A 50-ft extension of the proposed breakwater would provide greater wave protection, however, this plan was cost prohibitive. The originally proposed improvements at Laupahoehoe Point are shown in Figure 9.

St. Paul Harbor, St. Paul Island, Alaska

44. St. Paul Island is the northernmost and largest island of the Pribilofs, located in the southeastern Bering Sea. St. Paul Harbor is located in a cove on the southern tip of the island. A berm breakwater was constructed at the site during the early 1980's but failed during storms of 1984. A new conventional breakwater (750 ft in length) was constructed in 1985 but is not of sufficient length to provide wave protection to vessels, particularly during storm events. Scouring around the breakwater head and sediment accretion along portions of the shoreline were also apparent since construction of the breakwater.

45. A 1:75-scale hydraulic model of St. Paul Harbor was designed and constructed to study wave and shoaling conditions in the harbor and to determine the optimum configurations of proposed improvements (Bottin and Mize

Figure 9. Originally proposed harbor improvements at Laupahoehoe Point, Hawaii

1988). Irregular waves with periods ranging from 6 to 16 sec and heights ranging from 7 to 25 ft were generated for test waves from five deepwater directions with swl's of +3.2- and/or +5.0-ft mllw.

46. Model tests were conducted for existing conditions and 59 test plan configurations. Variations entailed changes in lengths, alignments, and crest elevations of breakwater extensions, breakwater spurs, and a secondary breakwater. The originally proposed improvement plan consisted of a 1,050-ft-long breakwater extension and an 800-ft-long vertical dock extension. Model tests for the proposed plan revealed wave heights of 6.8 ft along the dock, which was more than double the sponsor's 2.5-ft wave height criterion. Sediment tracer tests indicated accretion along the shoreline in the cove and deposits near the head of the new breakwater. The installation of several spur and/or secondary breakwater plans also resulted in excessive wave heights (3.5-ft maximum) along the vertical-wall dock. Some plans reduced wave heights to an acceptable level, but these plans were ruled unacceptable due to their narrow entrance channel conditions which would inhibit navigation.

47. Based on the results of the model study, a pile-supported dock, versus the vertical-wall dock, was recommended. It also was recommended that the 2.5-ft criterion along the duck be relaxed during the most severe storm events provided that vessels moved to other locations in the harbor which provided acceptable wave protection.

48. The optimum improvement plan tested in the model considering wave protection, navigation, harbor circulation, and costs included the original 1,050-ft-long breakwater extension with a detached 1,100-ft-long secondary breakwater (Figure 10). The secondary breakwater provided wave heights of 2.5 ft or less in its lee. It was determined that the structures would have no adverse impact on the sediment movement in the area, nor would shoaling be induced in the harbor entrance or mooring areas.

Siuslaw River, Oregon

49. The mouth of the Siuslaw River empties into the Pacific Ocean west of Eugene, OR. The mouth of the river is protected by jetties, however, shoaling occurs and frequent dredging of the entrance was required. Extensions of the existing jetties were authorized in 1981.

50. A 1:100-scale model study of the Siuslaw River project was conducted to qualitatively determine shoaling conditions at the river mouth for various test plans (Bottin 1981). Due to limited funds and time for the project, testing of the proposed jetty modifications was conducted on an existing

Figure 10. Optimum configuration for St. Paul Harbor, Alaska

model of Rogue River, Oregon, which had similar offshore contours. Test waves with periods ranging from 9 to 13 sec and heights ranging from 7 to 27 ft were generated from two deepwater directions for swl's of 0.0- and/or +6.7-ft mllw. Seven improvement plans were tested which included the original jetty extensions and various spur arrangements attached to one or both of the extensions.

51. The original test plan consisted of 1,900- and 2,300-ft extensions of the existing north and south jetties, respectively. With the jetty extensions in place, model tests indicated that sediment would move into the entrance for waves from both northerly and southerly directions. To alleviate sediment transport into the entrance channel, spurs were installed on the jetty extensions. Model tests indicated that 400-ft-long spurs installed 900 ft shoreward of the heads of the north and south jetty extensions (Figure 11) were optimum relative to prevention of shoaling in the entrance.

Port Orford, Oregon

52. Port Orford is situated on the Pacific Coast about 50 miles north of the Oregon-California border. The original harbor at Port Orford was located in a natural cove, protected from waves from the North and West. However, wave action from southwesterly winter storms frequently caused extensive damage to harbor facilities. Local interests constructed a breakwater

Figure 11. Optimum location of jetty spurs for prevention of shoaling at Siuslaw River, Oregon

that was only partially effective with respect to wave protection, and an extension was subsequently constructed. After completion of the breakwater extension, the harbor area adjacent to the pier began to shoal and extensive dredging was required.

53. A 1:100-scale hydraulic model of Port Orford Harbor was designed and constructed to study shoaling conditions in the harbor and to develop remedial plans to alleviate shoaling at the pier without significantly increasing wave action at the pier (Giles and Chatham 1974). Waves with periods ranging from 7 to 17 sec and heights ranging from 3 to 21 ft were generated from six deepwater directions with swl's of 0.0- and/or +7.3-ft mllw.

54. Model tests were conducted for prebreakwater conditions, existing breakwater conditions, and 53 variations to a range of improvement plans. Tests for prebreakwater conditions indicated wave heights along the pier in excess of 23 ft but no sedimentation at the pier. The existing breakwater drastically reduced wave heights at the pier for most d rections, but resulted in sediment deposits in the harbor area for all directions due to altered longshore currents and a large eddy in the harbor.

55. Original modifications entailed removing portions of the existing breakwater which were generally unsuccessful. Wave heights increased at the pier significantly with only a slight reduction in shoaling. After testing of

numerous additional breakwater configurations, an 1,100-ft-long structure (Figure 12) was determined to prevent shoaling by waves from any direction, and material already in the harbor remained stable (did not move toward the pier). In addition, wave heights in the harbor were not increased.

Figure 12. Breakwater configuration developed for Port Orford Harbor, Oregon, for elimination of shoaling

Rogue River, Oregon

56. The Rogue River enters the Pacific Ocean on the Oregon coast about 30 miles north of the California border. Two jetties, spaced 1,000 ft apart, were constructed to provide protection to the river mouth and to improve natural flushing of the navigation channel. A small-boat basin, protected by

a breakwater, was located on the south bank of the river. A persistent shoaling problem existed between the two jetties, along the inside of the south jetty, and in the turning basin and harbor access channel. Maintenance dredging was difficult, and the navigation channel was blocked which restricted vessel traffic between the ocean and port facilities.

57. A 1:100-scale hydraulic model was designed and constructed (Bottin 1982b, Bottin 1983) to study wave, shoaling, current, and riverflow conditions in the lower reaches of the Rogue River for existing conditions and numerous improvement plans. Test waves with periods ranging from 5 to 17 sec and heights ranging from 11 to 27 ft were generated from four deepwater directions for swl's of 0.0-, +1.5-, +4.3-, and/or +6.7-ft mllw. Maximum ebb and flood tidal flow conditions were simulated in the model as well as river discharges ranging from 50,000 t 350,000 cfs.

58. Model tests were conducted for existing conditions and 59 variations to several test plans. Improvement plans consisted of dikes installed within the existing entrance, jetty extensions (existing alignment, toward the west, and toward the south) with and without spurs, an alternate harbor entrance south of the river mouth, and reorientation of the mouth of the river with a decrease in the width of the entrance.

59. Model tests for existing conditions indicated that shoaling will occur in the lower reaches of the river for various test waves and swl's for each wave direction. Generally, material was deposited in the southern portion of the river adjacent to the south jetty and then migrated upstream across the entrance to the small-boat basin similar to conditions observed in the prototype. Dikes extending from the south jetty were oriented in a configuration that would prevent shoaling of the small-boat entrance; however, this resulted in increased water-surface elevations upstream of the dikes. Tests revealed that several jetty extension plans (with spurs) would prevent sediment from entering the river entrance from the north and south shorelines, but sediment moving down the river would eventually result in a shoal that would extend upstream to the small-boat basin entrance. The narrower, reoriented river entrance resulted in shoals adjacent to the new structures that could restrict or prohibit navigation and substantial increases in water-surface elevations in the lower reaches of the creek. Of all the improvement plans tested, a new entrance south of the existing river mouth (Figure 13) provided wave and shoaling protection from all sources.

Figure 13. A new entrance south (right) of the existing Rogue River mouth provides wave and shoaling protection for the harbor

Crescent City Harbor, California

60. Crescent City Harbor is located on the Pacific Ocean about 17 miles south of the Oregon border. The harbor entails an outer breakwater extending 4,670 ft from the shore on the west side of the harbor, a 1,200-ft-long inner breakwater attached to Whaler Island, and a rubble-mound barrier about 2,400 ft long to prevent sand movement into the inner harbor. The harbor is exposed to large incoming waves that cause damage to moored vessels and lost time for vessels because of undesirable conditions.

61. A 1:125-scale hydraulic model investigation was conducted (Senter and Brasfeild 1968) to determine the optimum length and location of an extension to the existing breakwater system that would reduce the adverse wave climate to tolerable levels in the harbor with respect to navigation and mooring conditions. Waves with periods ranging from 7 to 16 sec and heights ranging from 4 to 22 ft were generated from four deepwater directions using a +7.5-ft swl mllw.

62. Model tests were conducted for existing conditions and 14 test plan configurations. Existing condition tests indicated severe conditions in the outer harbor with wave heights up to 18 ft measured in the navigation entrance and wave heights greater than 3 ft in the harbor. The original improvement plan entailed a 2,000-ft-long extension of the outer breakwater; however, test results revealed that the established 2-ft design criterion would not be met in the inner harbor. Model tests indicated that the criterion in this area

could be met with a 400-ft-long extension of the inner breakwater. Test results showed that a 1,200-ft-long breakwater extending seaward from Whaler Island (Figure 14), would offer improved navigation and mooring conditions in the overall harbor, but there would be periods when the design criterion would be exceeded. It was also determined that a rubble-mound absorber, installed parallel to and on the harbor side of the existing outer breakwater, would provide adequate protection from waves that overtop the present structure.

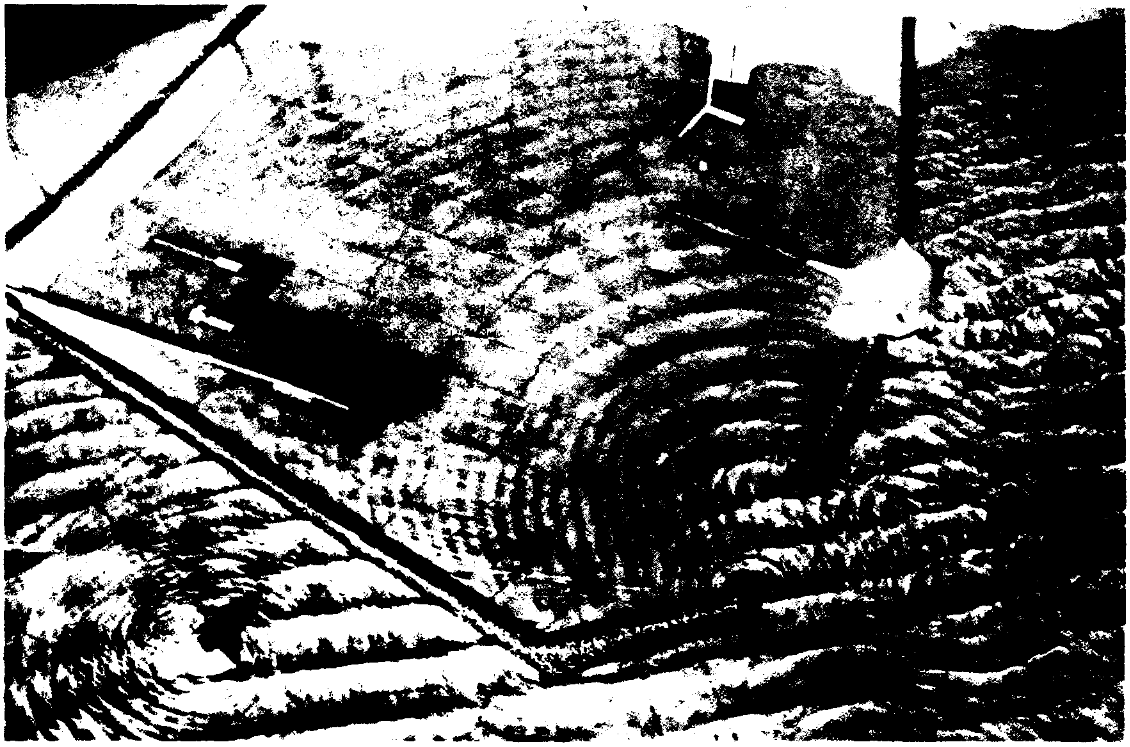

Figure 14. Breakwater configuration at Crescent City Harbor, California, provides improved overall conditions in the harbor

Noyo Harbor, California

63. Noyo Harbor is located on the California coast at the mouth of Noyo River approximately 135 miles north of San Francisco. The river empties into Noyo Cove which is exposed to large waves from ocean storms. Waves up to 14 ft high have been observed in the cove, and conditions in the jettied river entrance are often impassable.

64. A 1:100-scale model was designed and constructed to determine storm-generated wave conditions along a proposed loading pier for seagoing lumber barges with the proposed breakwater configurations installed (Wilson 1967). Waves with periods ranging from 11 to 17 sec and heights ranging from

14 to 26 ft were generated from four deepwater directions for a +6.9-ft swl mllw.

65. Tests were conducted for existing conditions and 28 test plan configurations. Model tests for existing conditions revealed wave heights up to 17 ft at the proposed pier location during storm wave events. The original project improvement plan included an 1,100-ft-long south breakwater and a 400-ft-long north structure. Test results, however, indicated inadequate wave protection throughout the cove for this plan. To obtain adequate wave protection at the pier (criterion of 2.0 ft established by the sponsor), it was determined that a 1,900-ft-long south breakwater in conjunction with a 320-ft-long north structure (Figure 15) would provide the desired wave protection in the cove. It was noted during testing that the use of rubble-wave absorbers around the entire perimeter of the cove were not economically justifiable based on the added wave absorption they provided.

66. Another hydraulic model investigation of Noyo River and Harbor was conducted subsequent to the above mentioned study. A 1:75-scale model was designed and constructed to determine the most economical breakwater configuration that would provide wave protection to the existing jettied entrance (Bottin, Acuff, and Markle 1988). Test waves with periods ranging from 7 to 19 sec and heights ranging from 6 to 32 ft were generated from five deepwater directions with +6.2- and/or +7.0-ft swl's mllw. In addition, long-period wave tests were conducted to determine the response of the small-boat harbor (located upstream of the entrance) to wave periods ranging from 60 to 200 sec. The deposition of riverine sediment in the entrance also was investigated for river discharges ranging from 7,000 to 41,000 cfs.

67. Model tests were conducted for existing conditions and 46 test plan configurations which included one or more breakwaters installed in the cove west of the entrance. Variations consisted of changes in the lengths, alignments, and locations of the structures. For a plan to be acceptable, the sponsor specified that maximum wave heights were not to exceed 6 ft in the entrance and the wave was to be nonbreaking.

68. Model tests for existing conditions revealed breaking wave heights in the entrance in excess of 10 ft for test waves from all directions. The original improvement plan consisted of a 370-ft-long breakwater. It resulted in wave heights in the entrance in excess of 8.5 ft. Various breakwater configurations, some with up to 1,125 ft in length, were tested, and it was determined that a 637-ft-long breakwater would result in entrance conditions

Figure 15. Optimum breakwater configuration at Noyo Cove, California, for wave protection of a seagoing barge loading pier

that met the established criterion. This configuration (Figure 16) did not interfere with the passage of riverine sediment through the entrance, and it resulted in improved long-period surge conditions in the river and harbor.

69. Additional tests were conducted in the 1:75-scale Noyo model to develop a breakwater plan for 14-ft design waves, as opposed to waves up to 32 ft previously tested (Bottin and Mize 1989). Thirty-one test plan configurations were tested with various combinations of inner and outer breakwaters, both attached and detached. The original test plan consisted of a 500-ft-long shore-connected outer breakwater and a 400-ft-long detached inner structure which resulted in wave heights well within the established criterion. Model tests indicated that a 250-ft-long inner breakwater alone would provide the required protection in the entrance for 14-ft design wave conditions from all directions.

Fisherman's Wharf,
San Francisco Bay, California

70. Fisherman's Wharf, located in San Francisco Bay near the Golden Gate, is an area bounded on the east by Pier 45 and on the west by the Municipal Pier. The area was essentially unprotected from wave damage.

Figure 16. Optimum breakwater configuration at entrance of No;ɔ River and Harbor, California, considering all wave conditions

Wave-energy from the open ocean (entering through Golden Gate) and local storms (waves generated by winds across the extensive water surface of the bay) resulted in continual damage to fishing vessels and mooring facilities.

71. A 1:75-scale hydraulic model was designed and constructed (Bottin, Sargent, and Mize 1985) to determine the most economical breakwater configuration that would provide adequate short period wave protection for small craft in the area. Test waves with periods ranging from 3.6 to 10 sec and heights ranging from 2 to 5.8 ft were generated from six directions with swl's of 0.0- and/or +5.7-ft mllw.

72. Model tests were conducted for existing conditions and 90 test plan variations which consisted in changes in the lengths, alignments, and locations of proposed solid, baffled, and/or segmented breakwater structures. Tests for existing conditions indicated wave heights up to 5.5 ft in the mooring areas of the harbor. Acceptance criteria wave heights, provided by the sponsor, were not to exceed 1.5 ft in the mooring area of a historic fleet of vessels (along Hyde Street Pier) and 1.0 ft in the small-craft mooring areas.

73. The originally proposed improvement plan consisted of a 1,450-ft-long solid outer breakwater constructed to form a 200-ft-wide entrance into the harbor. This configuration was ineffective with wave heights in excess of 4 ft measured in the small-craft mooring areas. A plan was developed that met the wave height criterion in the harbor. It consisted of a 1,585-ft-long outer solid breakwater which formed a 165-ft-wide entrance, and

a cumulative 585-ft length of baffled breakwaters attached to Pier 45. Further examination of this plan indicated excessive wave heights (9.0 ft) in the entrance due to reflected wave energy off the baffled structures. An optimum breakwater configuration was developed which consisted of a 1,585-ft-long solid outer breakwater positioned to form a 165-ft-wide entrance, and segmented breakwaters installed at Pier 45 with a cumulative length of 400 ft (Figure 17). The segmented structures had 28-ft solid sections and 6-ft openings which allowed tidal circulation through the outer harbor area. This plan met the wave acceptance criteria in the mooring areas with wave heights of 4.5 ft in the entrance during periods of storm wave attack.

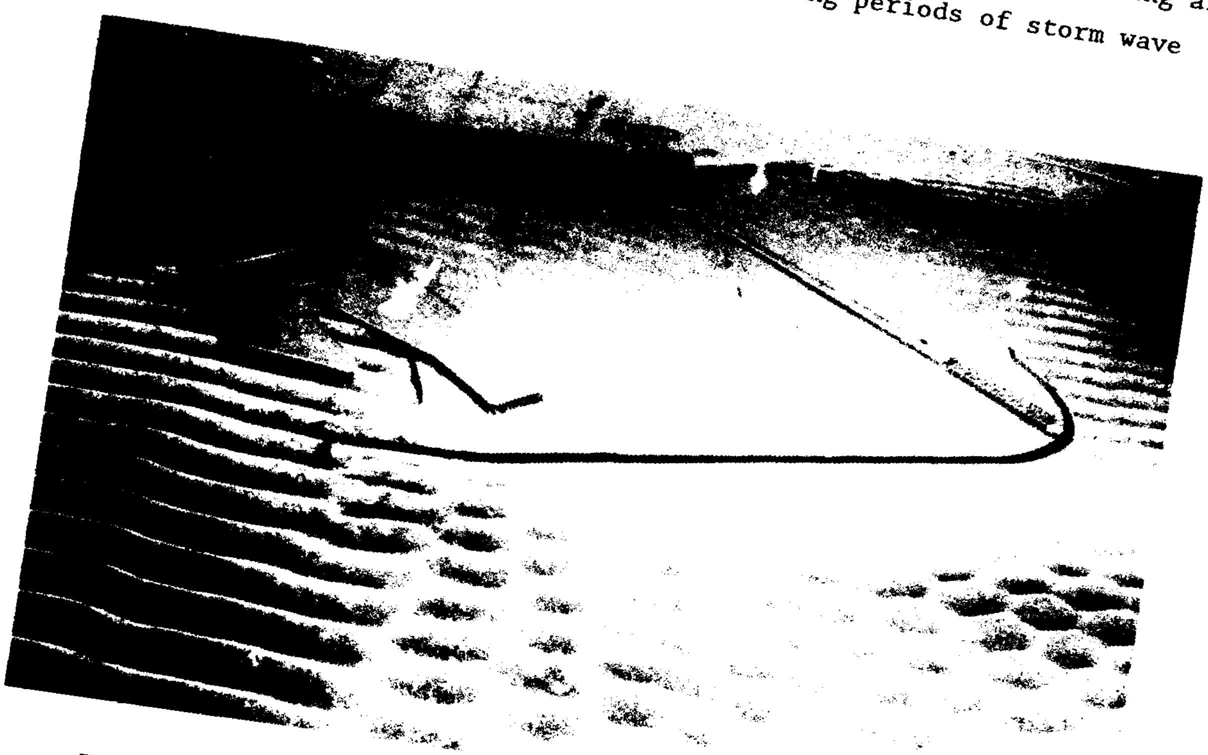

Figure 17. Optimum breakwater configuration for Fisherman's Wharf Area, San Francisco Bay, California

<u>Half Moon Bay Harbor, California</u>

74. Half Moon Bay Harbor, is located on the Pacific Coast about 20 miles south of San Francisco Bay. The harbor is about 1 mile long and 1/2 mile wide. It was protected by a 2,620-ft-long west breakwater and a 4,420-ft-long east structure. A 600-ft-wide navigation entrance provided access to the harbor for small fishing craft. Storm waves approaching from west counterclockwise to south-southeast entered the harbor through the

entrance and the transmission of wave energy through the voids of the existing breakwater frequently caused the berthing areas to be unusable.

75. A 1:100-scale hydraulic model of the harbor was designed and constructed to determine the optimum plan for reducing adverse wave action occurring in the harbor (Wilson 1965). Waves with periods ranging from 5 to 15 sec and heights ranging from 6 to 21 ft were generated from six deepwater directions for a +6.1-ft swl mllw.

76. Model tests were conducted for existing conditions, 11 breakwater plan configurations consisting of extensions to the heads of both the east and west structures, and the installation of additional offshore breakwaters. Tests for existing conditions indicated that wave heights in the inner harbor in excess of 2 ft would occur up to 20 percent of the time, and waves ranging from 6 to 7 ft would occur during severe wave attack. The wave height criterion adopted in the berthing area by the sponsor was not to exceed 2 ft for more than a few hours per year.

77. The originally proposed improvement plan consisted of a 400-ft-long extension of the west breakwater, reducing the entrance opening from 600 to 200 ft in width. Wave heights measured in the berthing areas exceeded the 2-ft criterion and little protection was afforded even with the narrowed entrance width. Further testing indicated that a 1,050-ft-long extension of the west breakwater positioned outside the existing harbor (seaward of the entrance) was optimum with regard to wave protection provided and economics. A view of the recommended configuration is shown in Figure 18.

Monterey Harbor, California

78. Monterey Harbor is located at the southern end of Monterey Bay and is about 100 miles south of San Francisco, CA. Wharfs were originally constructed at the site to provide support to the local fishing fleet. Since they were fully exposed to the weather, a 1,700-ft-long breakwater was subsequently constructed. This structure did not provide sufficient mooring area for the large number of boats in the harbor, and waves occasionally caused damage to vessels and harbor facilities and mooring difficulties for small craft in exposed areas of the harbor.

79. A 1:120-scale hydraulic model of Monterey Harbor was designed and constructed (Chatham 1968) to determine whether proposed harbor revisions would provide adequate protection from both long- and short-period wave and surge action. Short-period waves with periods ranging from 7 to 17 sec and heights ranging from 7 to 13 ft were generated from five deepwater directions

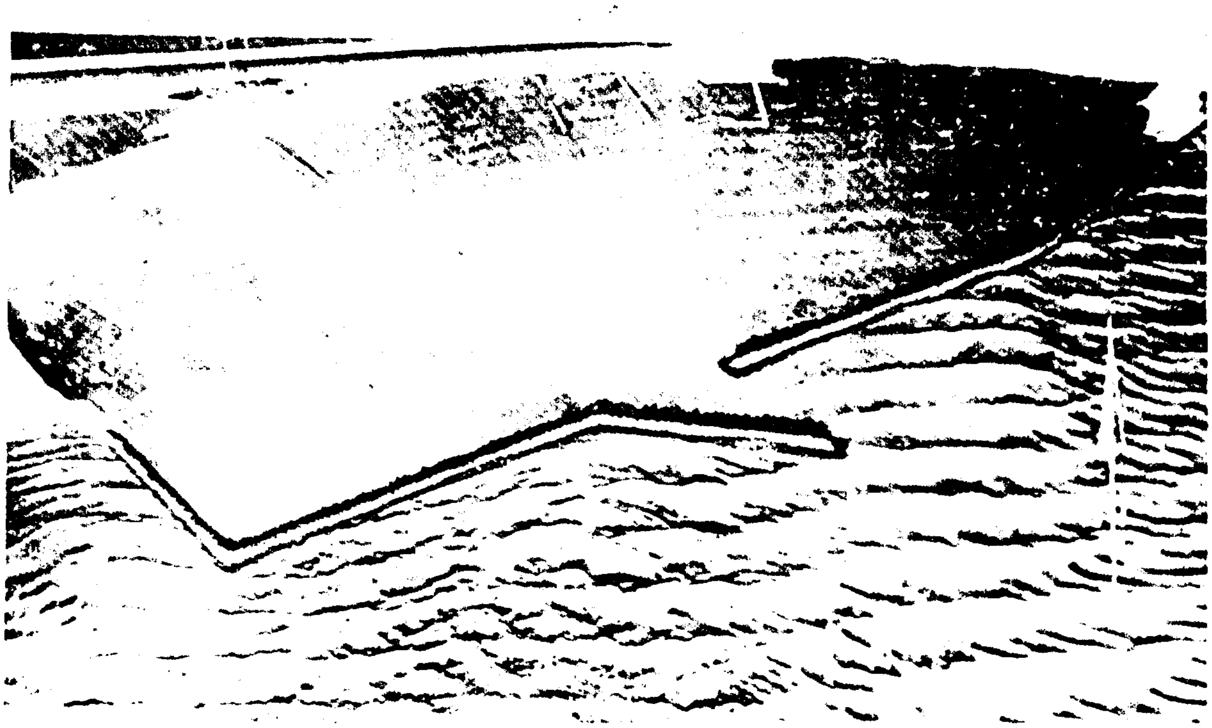

Figure 18. Optimum breakwater configuration recommended for Half Moon Bay Harbor, California

with an swl of +5.2-ft mllw. Long-period waves ranging in period from 35 to 255 sec were also reproduced in the model. A maximum wave height criterion of 1.5 ft was established by the sponsor in the inner basin for short-period tests. It was assumed that the proposed improvement plans would be satisfactory if long-period wave heights in the existing and proposed basins did not exceed those that occur in the existing marina.

80. Tests were conducted for existing conditions and two basic harbor configurations (a double entrance and a single entrance). Reductions in the length of the offshore structure also were tested for the double-entrance plan. Short-period wave height tests for existing conditions revealed wave heights in the berthing areas in excess of 5 ft during storm wave conditions. It was concluded from test results that both the single-entrance and the double-entrance plan would provide sufficient protection to the inner basins for short-period waves, and neither plan appeared to be significantly better than the other. Reducing the length of the offshore breakwater would have little effect on wave heights in the basins but would result in significant increases in wave heights in the east entrance to the harbor. It was determined from the investigation that long-period wave conditions in the harbor would be about the same for both proposed configurations, and either plan

would offer a slight improvement over conditions in the existing harbor. A model view of the double-entrance plan is shown in Figure 19.

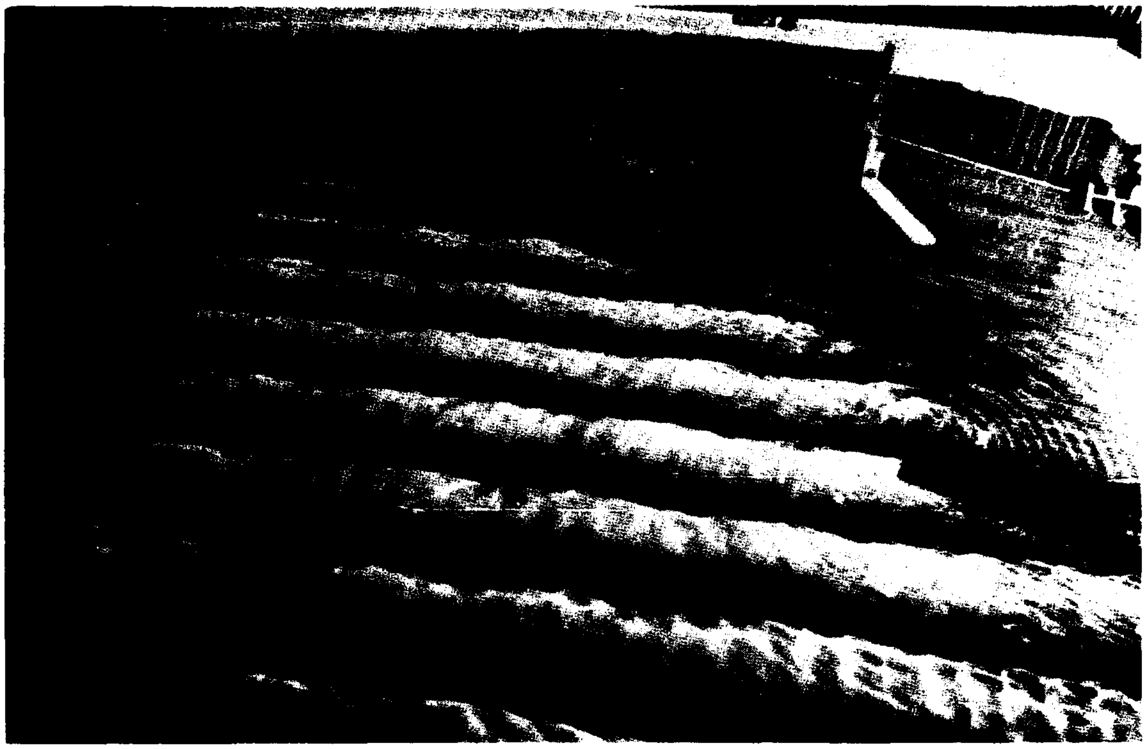

Figure 19. View of double-entrance breakwater configuration proposed for Monterey Harbor, California

Port San Luis, California

81. Port San Luis is located at the western end of San Luis Obispo Bay on the coast of southern California about 190 miles northwest of Los Angeles. An existing breakwater extended 336 ft from Point San Luis to Whaler Island and extended 1,820 ft seaward for a total length of 2,400 ft (including the island). The harbor was exposed to waves as high as 19 ft. Adverse wave action had caused damage to small craft in the harbor, and all recreational small craft were removed from the harbor each fall because of the danger of winter storms. Recreational boating was, therefore, restricted to about 8 months a year.

82. A 1:100-scale hydraulic model investigation of Port San Luis (Chatham and Brasfeild 1969) was conducted to study wave action in the harbor for proposed breakwater configurations. Waves with periods ranging from 7.5 to 14 sec and heights ranging from 7.5 to 18 ft were reproduced from five deepwater directions with an swl of +6.0-ft mllw. For a plan to be

acceptable, it was specified by the sponsor that waves were not to exceed 2.5 ft in the boat mooring area and 1.5 ft in the slip areas.

83. Tests were conducted for existing conditions and 26 test plan configurations. Test results for existing conditions revealed wave heights up to 19 ft in the proposed mooring area, indicating that considerable protection was needed for suitable anchorage and mooring of small craft. The originally proposed improvement plan consisted of an 1,150-ft-long south breakwater with a 370-ft-long wing extending northward from the structure, a 3,515-ft-long detached breakwater, and a 1,300-ft-long north breakwater that was shore-connected. The plan resulted in significant wave height reductions in the harbor (up to 85 percent when compared to existing conditions).

84. Numerous modifications were made to the originally proposed breakwater configuration. Tests indicated that the proposed 1,300-ft-long north breakwater and the 370-ft-long south breakwater wing could be replaced by revetted fills. The detached breakwater orientation was adequate, but the crest elevation could be reduced on the southern portion, and the south breakwater was increased to 1,550 ft in length. A view of the optimum improvement plan is shown in Figure 20. Except for a very small percentage of the time, wave height acceptance criteria should be met with the recommended plan in place.

Santa Barbara Harbor, California

85. Santa Barbara Harbor is located on the Southern California coast about 90 miles northwest of Los Angeles. The harbor is in the lee of an 1,800-ft-long rubble-mound breakwater, covers about 44 acres in area, and accommodates about 700 small-craft vessels. The harbor is exposed to direct wave action from several deepwater directions. Sediment migrates along the existing breakwater and is deposited in the lee of its eastern end and constant dredging is required. Plans were formulated to expand and improve the small-craft harbor by enclosing the area by a breakwater system.

86. A 1:100-scale hydraulic model was designed and constructed (Brasfeild and Ball 1967) to investigate wave action in the harbor for the proposed breakwater configurations. Waves with periods ranging from 8 to 16 sec and heights ranging from 8 to 18 ft were reproduced from four wave directions using a +6.0-ft swl mllw. For a plan to be acceptable, the sponsor specified that wave heights were not to exceed 1.5 ft in the small boat basin mooring areas.

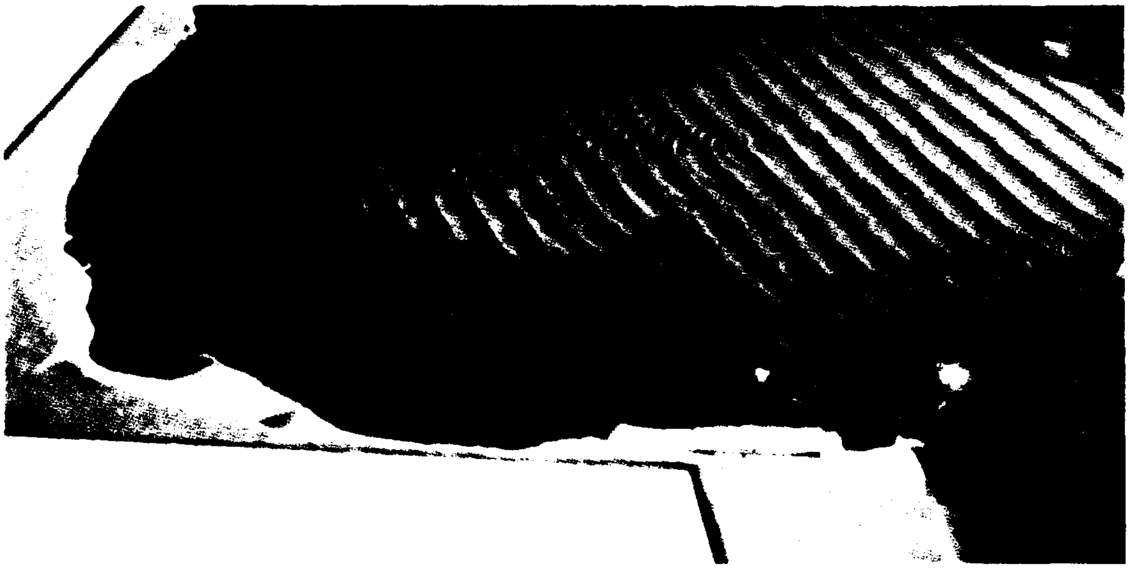

Figure 20. Optimum breakwater configuration developed for Port San Luis, California

87. Tests were conducted for 16 test plan variations. The originally proposed improvement plan consisted of a 500-ft-long extension of the existing (west) breakwater, a 3,600-ft-long shore-connected east breakwater, a 1,600-ft-long detached breakwater, a sand trap area, and three moles in the harbor that provided four mooring basins. Variations consisted of changes in the lengths, alignments, locations, crest elevations, and slopes of the breakwaters.

88. The wave height criterion was met for the original breakwater configuration; however, based on test results, modifications were made to the structures that significantly reduced costs. As a result of the study, the detached breakwater was reduced from 1,600 to 1,400 ft in length, the crest elevation of the west breakwater extension was reduced from +16 to +12 ft, the crest elevation of the westward 800-ft length of the detached breakwater was reduced from +16 to +12 ft, and the seaward slope at the corner of the east breakwater was flattened to reduce wave runup and eliminate overtopping that occurred at that location. A view of this plan configuration is shown in Figure 21. In addition, it was determined that the detached breakwater could be extended to 1,735 ft in length, with the west end reoriented to provide a larger sand trap and still provide adequate wave protection in the harbor basins.

Figure 21. Optimum breakwater configuration at Santa Barbara, California

Ventura Harbor, California

89. Ventura Harbor is located on the California coast approximately 55 miles northwest of Los Angeles. The harbor is entirely man-made and consists of three mooring basins and extensive land area which totals about 275 acres. Also included are a 1,500-ft-long offshore breakwater, north and south jetties with 1,250- and 1,070-ft lengths, respectively, and a 250-ft-long middle jetty. During most of the year, sand migrates along the beaches into the sand trap (located in the lee of the offshore breakwater) and entrance channel. Shoaling of the entrance results in frequent maintenance dredging and creates hazardous navigation conditions due to breaking waves and shallow depths.

90. A 1:75-scale hydraulic model was designed and constructed (Bottin 1991) to investigate wave and shoaling conditions at the harbor entrance for proposed modifications. Irregular waves with periods ranging from 8 to 17 sec and heights ranging from 6 to 15 ft were reproduced from five directions using swl's of +3.0- and/or +7.0-ft mllw. A granulated coal tracer material was used to qualitatively represent the movement of tracer material.

91. Model tests were conducted for existing conditions and 11 test plan configurations. Improvement plans consisted of an extension of the existing detached breakwater, a new south beach groin, a spur groin extending from the existing north jetty, and additional channel dredging. Variations entailed changes in the length, alignment, and location of the proposed north spur groin. Testing of existing conditions indicated excessive wave heights in the entrance channel for waves from westerly direction and very heavy tracer deposits in the entrance channel for waves from the northerly and southerly directions.

92. The originally proposed detached breakwater extension was effective in reducing wave heights in the entrance to acceptable levels, and the proposed south beach groin provided shoaling protection to the entrance for waves from the south. The proposed spur groin (250 ft long) attached to the existing north jetty, however, resulted in heavy deposits in the entrance channel for waves from northwesterly directions. Model tests revealed that the spur groin could be reoriented and lengthened (300 ft long) to minimize shoaling of the entrance channel. The spur groin created eddies over the deposition basin in the lee of the offshore breakwater in which most of the sediment deposited. A view of the optimum plan is shown in Figure 22.

Port Hueneme, California

93. The Port of Hueneme is situated on the coast of Southern California, approximately 65 miles northwest of the Los Angeles-Long Beach port complex. Harbor dredging and construction of dock facilities were completed in 1940 and were taken over by the Navy in 1942. In 1961 the original wharf and adjacent land was sold by the Navy to the Oxnard Harbor District. The harbor was dredged through the shoreline and had an east and west breakwater which protected the entrance. To accommodate rapid growth, the harbor commission proposed an expansion to the east basin of the harbor.

94. A 1:100-scale hydraulic model investigation was conducted to determine the effects of the proposed revision on mooring conditions in the harbor resulting from long-period waves (Crosby, Durham, and Chatham 1975). Prototype wave periods ranging from 30 to 230 sec at 5-sec intervals were generated from one direction (representing several deepwater directions) with a +5.4-ft swl mllw.

95. Tests were conducted for existing conditions and the proposed basin expansion which consisted of a 710- by 450-ft-rectangular extension to the existing east basin. A comparison of test results for existing conditions and

Figure 22. Sediment tracer patterns for optimum improvement plan at Ventura Harbor, California

the basin extension revealed that the proposed basin will not alter the major oscillation patterns of the harbor for wave periods less than 130 sec, and for periods greater than 130 sec, effects should be minimal. Wave heights versus frequency response in the expanded harbor will generally be equal to or less than those for the existing harbor. The model also indicated that expansion of the east basin would not produce significant changes in mooring conditions encountered in the present harbor. A general view of existing conditions in the model is shown in Figure 23.

Marina del Rey, California

96. Marina del Rey is a small-craft harbor located in Santa Monica Bay about 15 miles southwest of Los Angeles, CA. The marina was developed by dredging a 2-mile-long channel and eight lateral basins off the main channel. The entrance was protected by two jetties extending about 2,000 ft into the bay. After construction of the harbor, waves entering the wide entrance channel reflected off the vertical concrete perimeter walls of the channel and resulted in intolerable wave conditions in several of the basins.

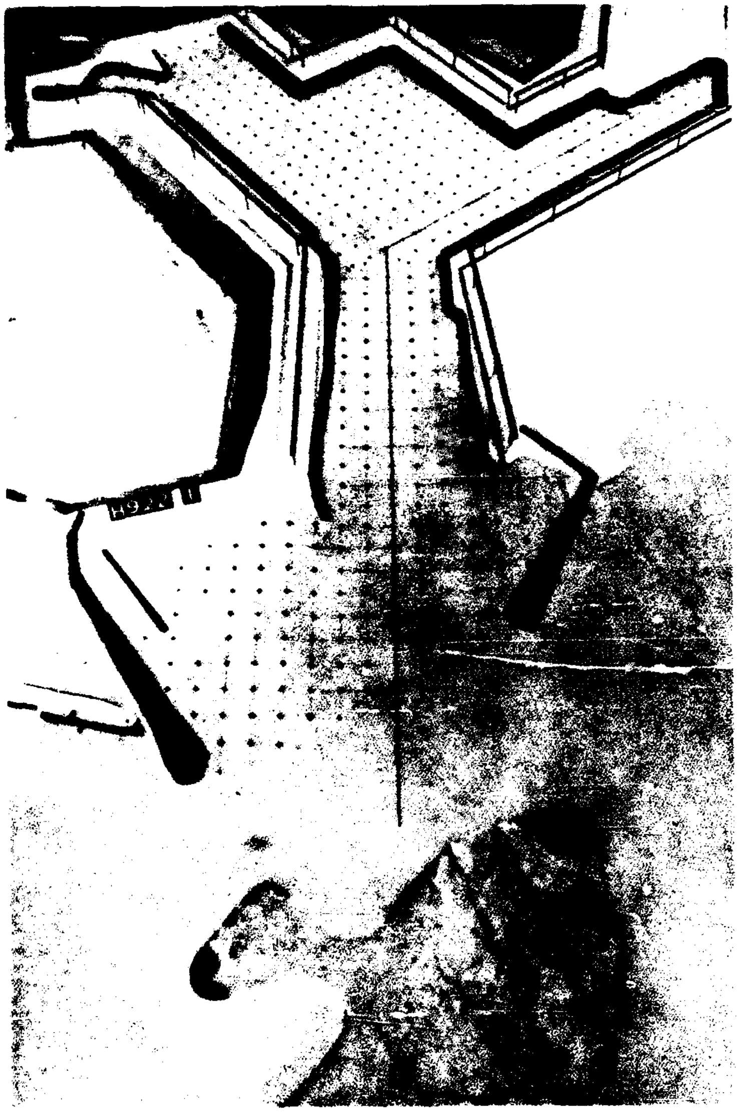

Figure 23. General view of existing conditions in the model of Port Hueneme, California

97. A 1:75-scale model investigation was conducted to determine the optimum improvement plan for reducing wave heights in the marina (Brasfeild 1965a). Acceptable wave heights in the harbor areas, established by the sponsor, were not to exceed 2 ft. Waves with periods ranging from 8 to 16 sec and heights ranging from 8 to 13 ft were reproduced from five deepwater directions for a +6.5-ft swl mllw.

98. Tests were conducted for existing conditions and 65 test plan variations. For existing conditions, wave patterns were very turbulent and confused in the channel and wave heights exceeded 10 ft in some of the interior basins. The initially proposed improvement plan consisted of a 2,000-ft-long

detached rubble-mound breakwater located 700 ft from the seaward ends of the existing jetties. Wave heights in the interior basins for this plan, however, ranged from 4 to 5 ft for some storm wave conditions. After testing numerous alternatives an optimum improvement plan was selected (Figure 24) that consisted of a 2,325-ft-long, wing-type offshore breakwater in front of the harbor entrance. In addition, it was recommended that the south jetty be sealed to an el of +8-ft mllw. This improvement plan, based on test results, would provide the desired reduction of wave action in the entrance, main channel, and individual basins by preventing approximately 95 percent of the short-period (sea and swell) wave energy from entering Marina del Rey.

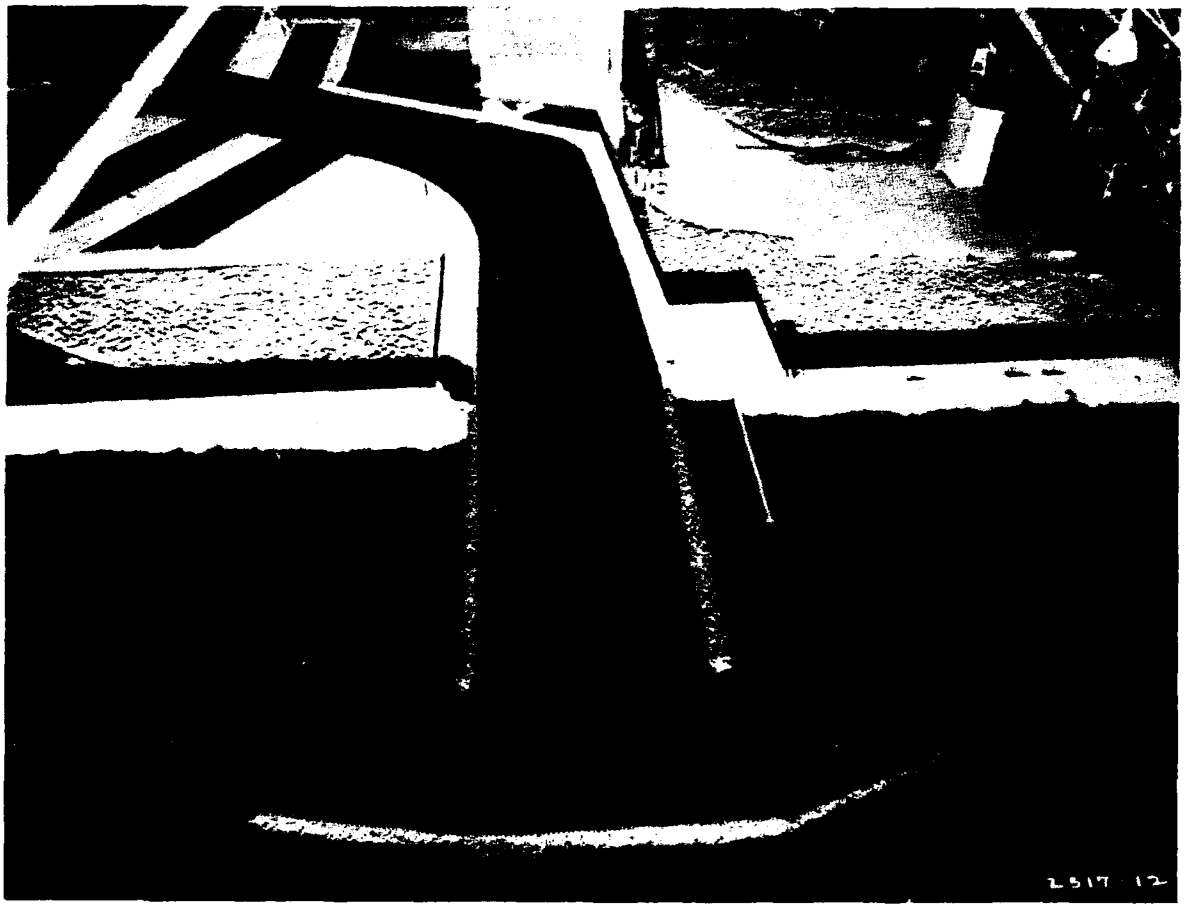

Figure 24. Optimum breakwater configuration developed for Marina del Rey, California

Redondo Beach King Harbor, California

99. Redondo Beach King Harbor is a small-craft harbor located on the Pacific coast at the southern end of Santa Monica Bay about 17 miles southwest of the business center of the City of Los Angeles. The harbor provides about

1,600 boat slips in three basins and is protected by a 4,285-ft-long north breakwater and a 600-ft-long south structure. The outer 1,600 ft of the north structure is low-crested (+14-ft mllw) while the remainder of the breakwater has a +20-ft crest el. During storms, energy of waves overtopping and transmitting through the structures and passing through the harbor entrance results in adverse conditions in the harbor, particularly in the lee of the low-crested section.

100. A 1:75-scale hydraulic model investigation was conducted to evaluate the adequacy of proposed improvement plans with regard to desired storm wave protection levels (Bottin and Mize 1990). Wave height criteria varied in the harbor for various return periods. Waves with periods ranging from 8 to 20 sec and heights ranging from 8 to 22.4 ft were reproduced by a spectral wave generator from three directions for swl's of +7.0- and/or +8.0-ft mllw.

101. Tests were conducted for existing conditions and 14 test plan configurations. Wave heights for existing conditions indicated very rough and turbulent wave conditions in the harbor with wave heights up to 8 ft along the moles for 50-year conditions (sponsor acceptance criterion was 3 ft for these conditions). The originally proposed improvement plan consisted of raising a 1,300-ft-long portion of the low-crested north breakwater to +20 ft and extending the south breakwater 300 ft. For this improvement plan, wave heights were in excess of the established criteria seaward of the moles. An optimum plan was developed which involved raising (to +20 ft) and sealing the 1,300-ft-long portion of the low-crested north breakwater. The structure was sealed by overlaying the harbor side of the breakwater with 200- to 2,000-lb stone gradation which was then capped with 11- to 19-ton armor stone. In addition, the south breakwater was extended 150 ft and a 300-ft-long portion around the dogleg was raised 4 ft (el +16). The optimum improvement plan is shown in Figure 25.

102. Additional tests were conducted in the 1:75-scale Redondo Beach King Harbor model to study wave conditions and determine the adequacy of proposed improvement plans in the northern portion of the harbor at Mole A (Bottin and Kent 1990). Test results for existing conditions indicated severe overtopping of the breakwater and subsequent flooding of Mole A. The original improvement plan entailed raising a 200-ft-long portion of the breakwater and flattening the slope of the structure on the sea side of Mole A. Model tests indicated that additional stone was required on an additional 150-ft-long portion of the breakwater to minimize overtopping and flooding of the mole.

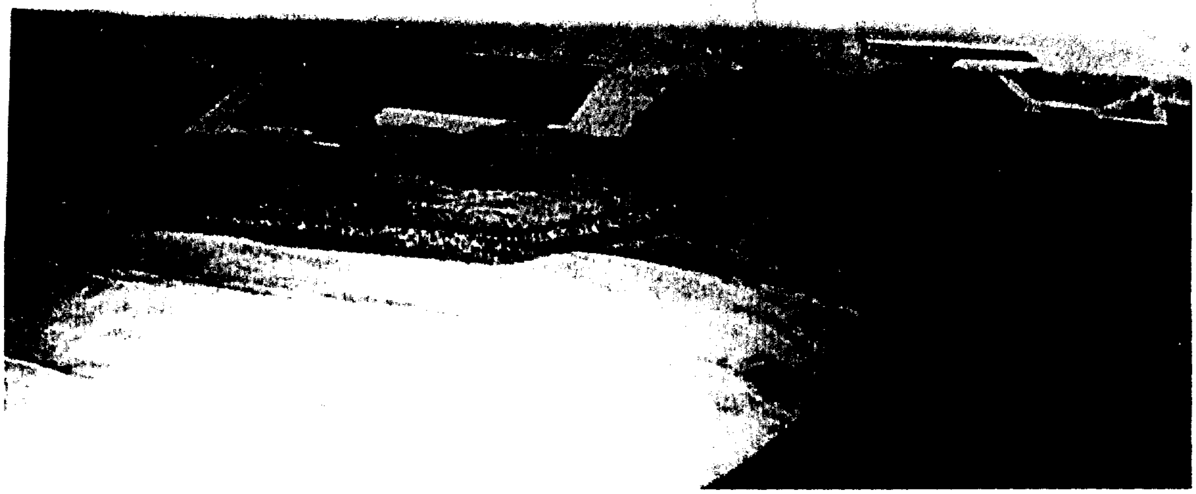

Figure 25. Optimum breakwater plan developed for Redondo Beach King Harbor, California

Fish Harbor, Los Angeles, California

103. Fish Harbor is located within Los Angeles Harbor, California, about 2 miles north-northwest of Angel's Gate. Many of the waterfront structures required replacement due to deterioration and obsolescence, and unfavorable wave conditions during storms prevented the expansion of small-boat moorings. Improvements proposed for the harbor included removing some of the existing breakwaters, deepening of the basins, excavating portions of land in the harbor, creating new landfills, and installing new breakwaters to provide protection for small craft during storm wave events.

104. A 1:60-scale hydraulic model was designed and constructed to determine breakwater modifications required to provide short-period wave protection for the proposed development project. Test-wave periods and heights ranging from 4 to 19 sec and 2 to 7 ft, respectively, were generated from two wave directions using a +5.4-ft swl mllw.

105. Tests were conducted for existing conditions and 18 test plan variations of the improvement plan. Test results for existing conditions revealed rough and turbulent wave conditions in the harbor with wave heights in excess of 4 ft in the mooring areas of the outer harbor and 3 ft in the inner harbor. For a plan to be acceptable, wave heights were not to exceed 1.5 ft in the commercial fishing basin and 1.0 ft in the recreational boating area. The original improvement plan resulted in wave heights in excess of 3.0 ft along a vertical-wall portion of the newly developed harbor. Various channel alignments, spur lengths, and absorbers were tested. After further

evaluation, a 200-ft-long breakwater spur attached to the breakwater at a point 700 ft seaward of the vertical wall (Figure 26) was developed as the optimal plan. It met the sponsor's established criteria and provided a future area of expansion for small craft in the lee of the breakwater.

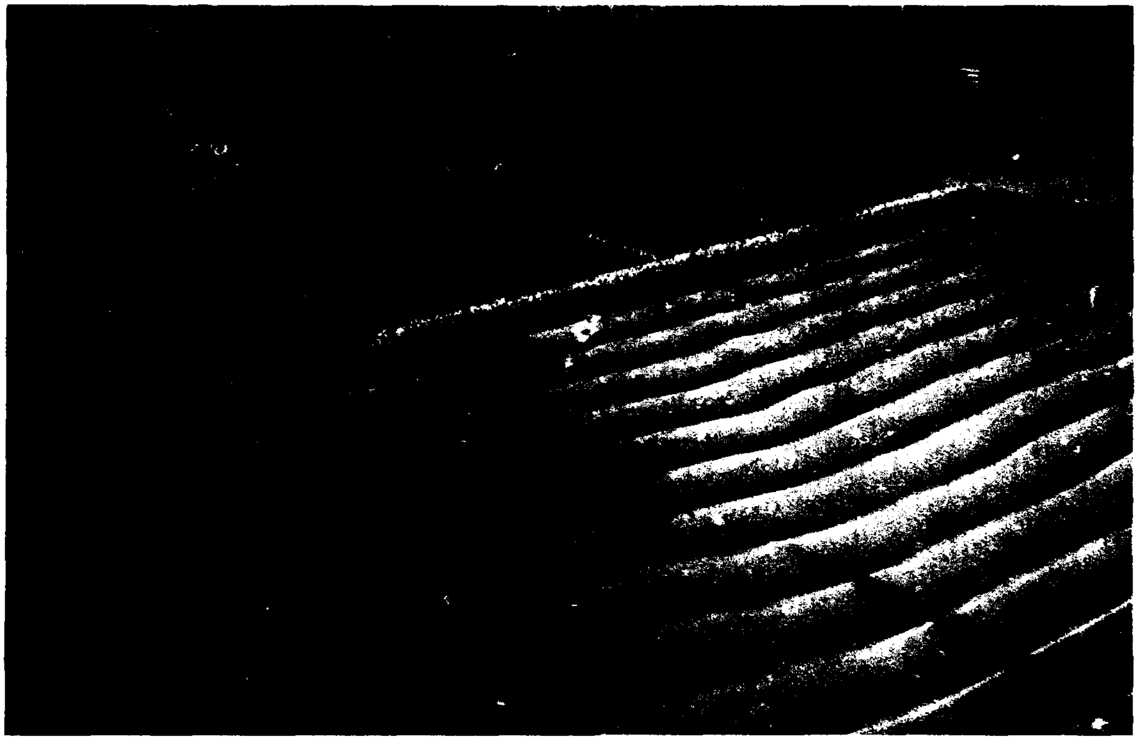

Figure 26. Optimum breakwater configuration for Fish Harbor, Los Angeles, California

Bolsa Chica Harbor, California

106. Bolsa Chica is located on the Pacific coast south of Long Beach, CA. An ocean entrance project with an associated marina complex was proposed at Bolsa Bay. Both navigable and nonnavigable ocean entrance concepts were being considered for construction.

107. A 1:75-scale hydraulic model investigation was conducted (Bottin and Acuff 1989) to determine wave penetration into the marina basin; to study circulation and sediment transport paths in the vicinity of the proposed structures; and to assess the entrance channel and jetty design configurations for both the navigable and nonnavigable ocean entrance concepts. In addition, the effects of flood flows entering the marina complex from Wintersburg Channel were determined. Waves with periods ranging from 5 to 17 sec and heights ranging from 7 to 15 ft were reproduced by a spectral wave generator for five

deepwater directions of wave approach using swl's of 0.9-, +2.8-, +3.0-, +7.0-, and/or +8.0-ft mllw. Maximum flood and ebb tidal currents were reproduced during the model testing program.

108. Tests were conducted for 18 variations of three basic improvement plans. The plans consisted of a proposed navigable entrance, both with and without a connector channel to Huntington Harbour, and a nonnavigable entrance in the vicinity of Bolsa Bay. Plans involving the proposed marina would be acceptable to the sponsor if the wave heights in the interior basins did not exceed 1.0 ft for wave conditions with 1-year recurrence intervals and 1.5 ft for waves with a 20-year recurrence interval.

109. The originally proposed improvement plan for the navigable ocean entrance with a connector channel did not meet the established wave height criteria. Model tests indicated the criteria would be achieved by the installation of rubble absorbers along the interior channels and a spur across the opening of one of the basins, in conjunction with raising the crest el of a portion of the offshore breakwater by 4 ft. A view of this plan is shown in Figure 27. The lengths of the north and south offshore breakwater wings were adequate to prevent shoaling in the entrance channel. It was also determined that Wintersburg Channel discharges should have minimal impacts in the interior basins of the marina complex.

Figure 27. View of entrance configuration for navigable entrance with a connector channel to Huntington Harbor for Bolsa Chica Harbor, California

110. The originally proposed improvement plan with the navigable ocean entrance without a connector channel did not meet the specified criterion in the interior basins. Variations were tested that made the plan acceptable. It was determined that removal of 750 ft or more of the north wing or 250 ft of the south wing of the offshore breakwater would result in sediment deposits in the entrance to the complex.

111. Tests conducted for the nonnavigable ocean entrance plan indicated 5- to 7-ft waves in the entrance during storm wave conditions. Some sediment along the shoreline and in the breaker zone will bypass the new entrance, and some will penetrate into the channel for waves from all directions.

Dana Point Harbor, California

112. Dana Point, California, located on the Southern California coast about 40 miles southeast of the Los Angeles-Long Beach Harbors, was the site of a proposed small-boat harbor. The harbor was proposed in a sheltered cove in the lee of Dana Point. After development, the proposed harbor would enclose an area of about 210 acres and provide berthing facilities for about 2,150 small boats. The cove is exposed to storm waves from directions ranging from southwest counterclockwise to south-southeast and to ocean swells from the south. The proposed harbor would be subjected to damaging wave energy reaching the berthing areas by entering through the outer navigation entrance and being transmitted through and/or overtopping the proposed rubble-mound breakwaters.

113. A 1:100-scale hydraulic model investigation was conducted to determine the optimum breakwater plan and location and size of the navigation opening that would provide adequate protection for the mooring areas during storm wave activity (Wilson 1966). Waves with periods ranging from 9 to 18 sec and heights ranging from 8 to 18 ft were generated from eight deepwater directions using an swl of +6.7-ft mllw. A wave height acceptance criterion of 1.5 ft was established in the harbor berthing areas by the sponsor and waves in the fairway were not to exceed 4 to 5 ft.

114. Tests were conducted for existing conditions and 13 test plans. Results for existing conditions indicated rough and turbulent conditions in the area even for low-magnitude storm waves. The proposed improvement plan involved construction of outer breakwaters and inner-harbor development consisting of east and west basin berthing areas partially enclosed by the shoreline on the north and a mole section on the south, southeast, and southwest

(Figure 28). Test results indicated that wave conditions in the berthing areas were acceptable, however, wave heights in the fairway were about 6.5 ft for severe storm wave conditions. It was noted that these conditions were due to a standing wave system caused by reflected waves from the mole slopes. Test results revealed that modifying the mole slope flanking the fairway, to include a berm, would reduce wave action considerably in the fairway.

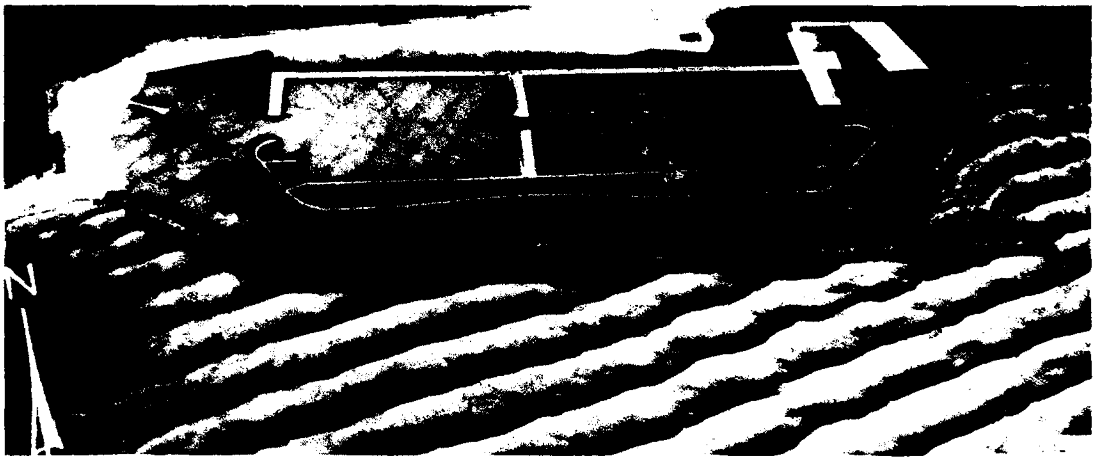

Figure 28. Model view of proposed harbor at Dana Point, California, under attack by storm waves

<u>Oceanside Harbor, California</u>

115. Oceanside Harbor, California, is located on the Pacific Ocean about 30 miles northwest of San Diego. The harbor complex consists of Del Mar Boat Basin (also known as Camp Pendleton Harbor) and the Oceanside Small-Craft Harbor. The harbors are protected by a 4,350-ft-long north breakwater and a 1,330-ft-long south jetty. After construction of Del Mar Boat Basin, persistent and devastating erosion of the beaches occurred with accompanying accretion of sand in the harbor and entrance channel. Proposals for the prevention of harbor shoaling included a 1,400-ft-long offshore breakwater and a 735-ft-long extension of the south jetty. Another plan was proposed which expanded the harbor facilities by converting the existing turning basin into an inner mooring basin. This plan included a 2,200-ft-long inner breakwater.

116. A 1:100-scale hydraulic model was design and constructed to investigate the arrangement and design of proposed structures for improving navigation and mooring conditions and preventing shoaling of the harbor (Curren and Chatham 1980). Waves with periods ranging from 7 to 19 sec and heights ranging from 4 to 16 ft were generated from four deepwater wave directions using

swl's of 0.0- and +5.4-ft mllw. With regard to wave conditions, wave heights in the berthing areas of the harbor expansion and in the expansion entrance were not to exceed 1.5 and 4.0 ft, respectively.

117. Tests were conducted for existing conditions and 88 test plan variations consisting of changes in the lengths and alignments of the breakwater structures and jetty extensions, changes in the north jetty cross section, the addition of an additional small-craft basin, and the construction of sand traps. Existing conditions revealed rough and turbulent wave conditions in the entrance channel due to waves breaking on the shoal across the harbor entrance, diffracting around the jetties, and overtopping the north jetty. Strong longshore currents were observed which contributed to hazardous entrance conditions. Tracer tests indicated that the model accurately reproduced general sediment patterns observed in the prototype.

118. Test results for the originally proposed improvement plan indicated that the offshore breakwater was ineffective in trapping sediment outside the harbor entrance and contributed to the shoaling problem by trapping material in the entrance channel. This plan was ineffective in reducing wave heights in the harbor expansion to the desired levels. Model tests indicated that stub groins and 500-ft radii sand traps located at strategic locations adjacent to the north breakwater and south jetty would minimize shoaling in the entrance for all wave directions and water levels. Extensions of the north breakwater and south jetty, with a stub groin installed on the south jetty, would result in wave heights within the established criteria. Typical wave and shoaling patterns at the harbor entrance are shown in Figure 29.

Mission Bay Harbor, California

119. Mission Bay Harbor is located on the coast of Southern California, about 10 miles north of San Diego Bay. The harbor entrance, leading to several coves and basins, was protected by two jetties (designated north and middle jetty) which extended into the bay about 3,800 and 4,600 ft, respectively. The harbor accommodated about 1,900 small boats, consisting entirely of recreational and sport fishing craft. The harbor is exposed to wind waves from deepwater directions, northwest counterclockwise through south. During storms (with incident waves larger than 10 ft in height) undesirable wave action and vessel damage occurred in Quivera Basin and Glen Rick Cove (located inland of the shoreward ends of the jetties).

Figure 29. Wave and shoaling patterns at Oceanside Harbor entrance for improvement plan with waves from south

120. A 1:100-scale hydraulic model investigation was conducted to determine wave effects in the existing harbor and to test and develop improvement plans proposed for reducing wave heights within Quivera Basin and Glen Rick Cove to satisfactory levels (Ball and Brasfeild 1969). Waves with periods ranging from 7 to 19 sec and heights of 9 to 19 ft were reproduced from seven deepwater directions using a +5.4-ft swl mllw.

121. Tests were conducted for existing conditions and eight improvement plans. For existing conditions, wave heights of 1.7 and 2.9 ft were obtained in Quivera Basin and Glen Rick Cove, respectively. The originally proposed improvement plan entailed impervious vertical structures that reduced the widths of both basins by 50 percent. This plan resulted in only slight wave height reductions in the basins. Model tests revealed that modifications were required to the south bank of the entrance channel. The existing curved portion of the bank line was revised to a series of right-angle steps that extended to the Quivera Basin entrance (Figure 30). This test plan resulted in an overall reduction in wave heights of approximately 79 percent in Glen Rick Cove and 23 percent in Quivera Basin.

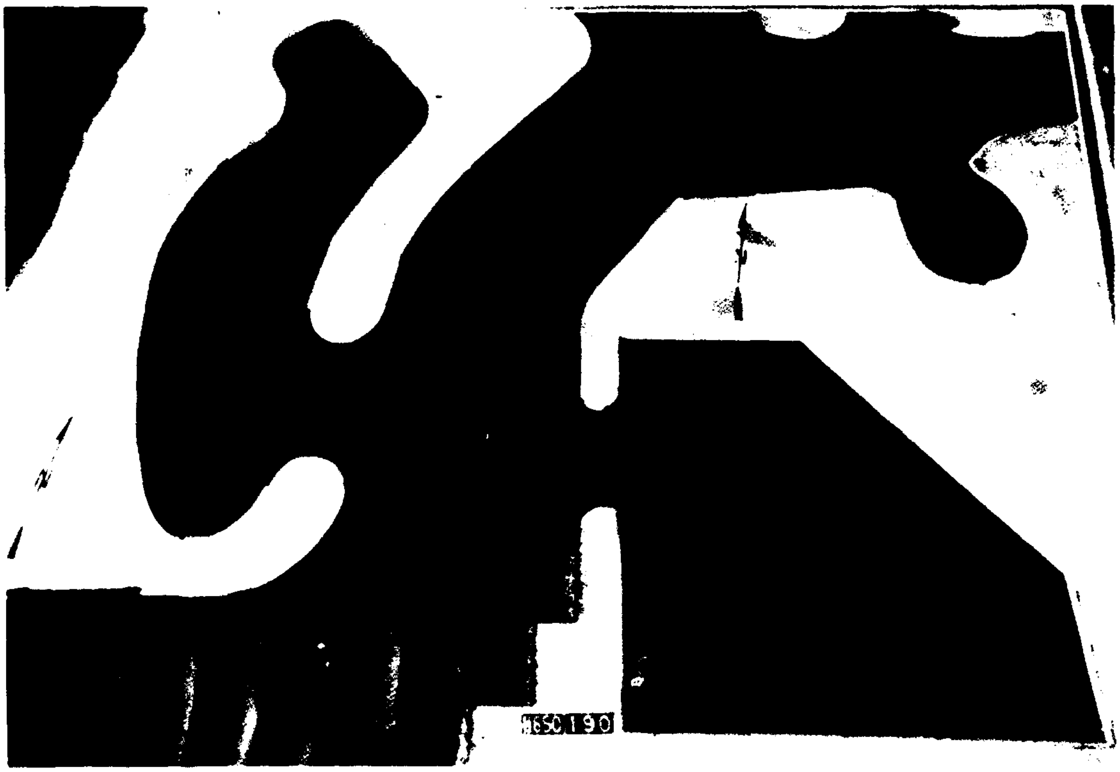

Figure 30. Optimum development of southern bankline at Mission Bay Harbor, California

122. Another hydraulic model investigation of Mission Bay Harbor was conducted subsequent to the previously mentioned study. A 1:100-scale model was designed and constructed to investigate the arrangement and design of proposed structures for improving hazardous entrance conditions and reducing surge inside the harbor (Curren 1983). Test waves with periods ranging from 7 to 19 sec and heights ranging from 5 to 15 ft were reproduced from three deepwater directions with swl's of 0.0-, +2.7-, and/or +5.4-ft mllw. In addition, long-period wave tests were conducted to determine surge conditions in the bay for wave periods ranging from 30 to 140 sec. Maximum flood and ebb tidal flows through the bay also were reproduced during the testing program.

123. Model tests were conducted for existing conditions and 30 test plan configurations which entailed changes in the lengths, alignments, and cross sections of proposed offshore breakwaters. For an improvement plan to be acceptable, the sponsor specified that maximum wave heights in the harbor entrance between the jetties were not to exceed 1.5 ft for deepwater waves of 6 ft or less. The originally proposed improvement plan consisted of a 2,200-ft-long offshore breakwater with a +22.5-ft crest el positioned 900 ft

seaward of the harbor entrance. The plan also included the removal of 220 ft from the end of the existing north jetty. Test results revealed the criterion in the entrance would not be met with this test plan. After evaluation of numerous alternatives a plan was developed (Figure 31) which consisted of a 1,600-ft-long offshore breakwater installed 525 ft offshore with a +17.5-ft crest el and an impervious core to a +7.5-ft el (original core was el -12.5 ft). Concurrent removal of 230 ft of the north jetty was included. This plan provided acceptable wave protection in the harbor entrance. In addition, long-period harbor oscillation tests indicated that the breakwater effectively reduced long-period energy in the harbor. In most cases, the selected improvement plan reduced resonant peaks by 50 percent or more when compared to existing conditions.

Figure 31. View of optimum entrance protection plan at Mission Bay Harbor, California

124. Additional tests were conducted in the second model for improving hazardous entrance wave conditions and reducing surge inside the harbor while minimizing impacts on surfing (Bottin and Acuff 1985). Wave heights in the entrance were relaxed from 1.5 to 4.0 ft for incident waves of 6 ft or less for these tests. Model tests were conducted for 10 variations in the design elements of three basic harbor configurations. The optimum plan, considering wave protection, ease of navigation, and economics, consisted of a

1,000-ft-long offshore breakwater with a +17.5-ft crest el (core el -12 ft) located 525 ft seaward of the jettied entrance. Concurrently, 230 ft of the north jetty was removed to facilitate navigation and water circulation. This plan resulted in significantly improved surge conditions in the harbor and should have minimal impact on surfing conditions on the beaches adjacent to the jetties.

San Juan Harbor, Puerto Rico

125. San Juan Harbor is located in San Juan Bay which is on the northern coast of the island of Puerto Rico. The US Navy proposed to establish a seaplane base in the outer harbor in the vicinity of the bay entrance. Protection from waves had to be provided just as it would for a small-boat harbor basin. Conditions in the outer harbor were unfavorable, particularly during periods when the bay was subjected to heavy swells from north and northeast. Ocean swells occurring during calm weather periods were the most detrimental to seaplane operation, however, local storms also generated waves that enter the bay and produced unfavorable wave conditions. For an improvement plan to be acceptable, the sponsor specified that waves were not to exceed 2 to 3 ft in the seaplane harbor.

126. A 1:100-scale hydraulic model was designed and constructed to determine the optimum location of breakwaters to afford adequate wave protection to the seaplane harbor (Bolin 1940). Waves with periods ranging from 17 to 30 sec and heights ranging from 8 to 24 ft were reproduced from four deepwater directions with an swl of 0.0-ft mllw.

127. Model tests were conducted for existing conditions and 17 test plan configurations. For existing conditions, wave heights of about 7 ft were recorded in the new harbor area. It was noted that smaller incident wave heights produced larger waves in the harbor since the most severe incident waves would break and lose part of their energy before reaching the area. Waves propagating through the San Juan Bay entrance are shown in Figure 32.

128. The original plan of improvement consisted of a 400-ft-long shore-connected breakwater (+13-ft crest el) at the entrance to San Juan Bay. Test results indicated this plan totally ineffective, and numerous offshore breakwaters, some with lengths up to 5,000 ft, were evaluated. The optimum plan entailed a 3,250-ft-long offshore breakwater with a +10-ft crest el. Personnel involved with the study thought the obvious solution would be breakwaters at the harbor entrance, but this turned out to be totally ineffective.

Figure 32. Storm waves at the entrance to San Juan Bay, Puerto Rico

Irregularities of the bathymetry transformed wave heights and directions and demonstrated the value of the model study.

Nassau Harbor, Bahamas

129. The port of Nassau, Bahamas, is situated on the northern coast of New Providence Island. The harbor is located in a wide, shallow channel between the city of New Providence and Paradise Island. The channel was about 250 ft wide and 25 ft deep. The harbor was exposed to wind waves from directions between northwest clockwise to northeast. Wave action and wave-induced currents in the harbor area were very hazardous to small craft and the entrance was almost impossible to navigate when high waves prevailed.

130. A 1:100-scale hydraulic model investigation was conducted to determine the optimum arrangement and design of certain proposed harbor improvements with respect to wave action and to determine current directions and velocities in the navigation channel and inner harbor area (Brasfeild 1965b). Waves with periods ranging from 5 to 11 sec and heights ranging from 5 to 15 ft were reproduced from three deepwater wave directions with a +5.0-ft swl mllw. Steady-state tidal flows also were simulated in the model through the harbor area.

131. Model tests were conducted for the existing site and 18 proposed harbor improvement plans. Variations to the plans consisted of changes in the locations and crest elevations of proposed structures. The originally proposed improvement plan consisted of constructing protective breakwaters at the channel entrance and constructing a large artificial island in a shallow area inside the new west breakwater. Due to time limitations, it was determined not to optimize the length and orientation of the two breakwaters, but to refine the structures as originally located. Model tests indicated the originally proposed harbor design was adequate in achieving the desired results. Results also revealed that refinements could be made that would reduce construction costs and still provide adequate protection. The crest elevations of the breakwaters could be lowered by 5 ft (+20 to +15 ft) provided wave absorbers were installed along the breakwaters at critical locations. Also, relocation of the artificial island was recommended in the study. A view of the plan recommended by the model test results is shown in Figure 33. Model test results verified that installation of the proposed revisions in the harbor will not result in intolerable current patterns or excessive velocities in regard to vessel operation.

Murrells Inlet, South Carolina

132. Murrells Inlet is located on the South Carolina shoreline approximately 15 miles southwest of Myrtle Beach. Prior to stabilization, the inlet was a natural channel through a sandy shoreline that maintained its existence due to currents generated by ocean tidal height variation. The inlet provided passage from the ocean to docking facilities for charter craft, commercial fishing vessels, and private boats. Due to the influx of sand into the inlet, an environment of shallow shifting-sand shoals, and breaking waves, difficult and dangerous navigation conditions existed at the inlet entrance. A project for the improvement and stabilization of Murrell's Inlet was authorized in 1971.

133. A hydraulic model investigation was conducted to determine the (a) optimum alignment of jetties and the spacing between them, (b) proper channel alignment, (c) current patterns in the entrance, (d) effects of improvements on the tidal prism and bay tidal elevations and velocities, and (e) wave heights in the entrance channel and deposition basin (Perry, Seabergh, and Lane 1978). The model was equipped with the necessary equipment to reproduce and measure tidal elevations, current velocities, and waves.

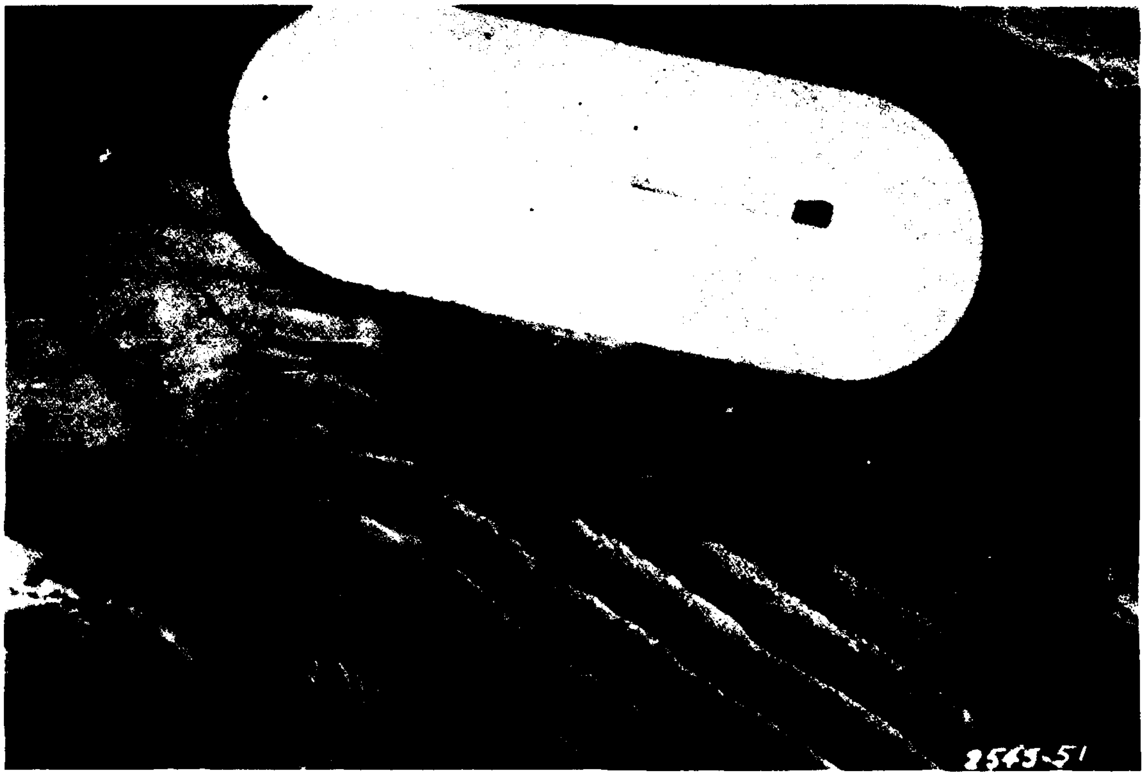

Figure 33. Recommended plan of improvement for Nassau Harbor, Bahamas

Prior to testing of improvements, the model tidal elevations and velocities were verified based on prototype data previously obtained.

134. The originally proposed improvement plan consisted of a 2,800-ft-long north jetty with a 1,300-ft low weir section, a 2,300-ft-long south jetty, two sand dikes, an entrance channel, an inner channel, and a deposition basin. Numerous modifications were made in the model and an optimum plan was developed that provided a stable entrance channel while minimizing other undesirable effects. The optimum plan included a 3,455-ft-long north jetty with a 1,330-ft low weir section, a 3,330-ft-long south jetty, two sand dikes, an entrance channel, an inner channel, a deposition basin, and a training dike. Figure 34 is a photo of the plan showing surface current patterns. Test results revealed that the recommended plan would have no significant impacts on tidal conditions and entrance velocity conditions, would be conducive to a self-maintaining channel, and would have no impact on the tidal prism.

Figure 34. Typical surface current patterns for optimum plan developed in the Murrells Inlet, South Carolina, hydraulic model

Little River Inlet, South Carolina

135. Little River Inlet was an unimproved tidal inlet along the state border of North and South Carolina. The inlet provided passage to the ocean from a sheltered bay for charter and commercial fishing boats and private recreational vessels. It also provided access to the Atlantic Intracoastal Waterway from the ocean. Dynamic charges in the position of the main ebb channel and inlet shoals were historically experienced within the inlet opening. The narrow navigable channel and shallow-bar regions resulted in

difficult and dangerous navigation conditions. Improvements for the inlet were authorized in 1972.

136. A hydraulic model investigation was conducted to determine the (a) optimum alignment of jetties and the spacing between them, (b) minimum length of jetties required, (c) proper channel alignment, (d) characteristics of the channel with respect to the influx of sediment at the entrance, (e) effectiveness of weirs on the jetties to pass longshore drift into the sedimentation basins, (f) effects on tidal prisms and bay tides, (g) effects on bay salinities, (h) wave heights in the entrance channel and deposition basin, and (i) location of sediment basins (Seabergh and Lane 1977). The model was equipped with all necessary appurtenances to reproduce and measure tidal elevations, current velocities, waves, freshwater inflows, sediments used in shoaling tests, and salinity. Prior to testing of improvement plans, the model was verified based on prototype data obtained.

137. The originally proposed improvement plan consisted of two jetties, sand dikes, a 300-ft-wide entrance channel through the offshore bar, and a 90-ft-wide inner channel. Modifications were made to the originally proposed configuration which included shortening of the jetty system and the installation of weirs on each jetty to trap longshore drift. Armoring various areas of the system also would prevent erosion at specific locations determined in the model study. Tracer tests indicated that the location, orientation, and elevations of the weirs and sediment basins were effective in permitting longshore sediment transport to pass over the weirs into the basin for the optimum plan, and the spacing between jetties was adequate to pass tidal flow without excessively high or undesirably low velocities. The improvement plan also resulted in no significant change to the salinity regimen of the bay. The optimum improvement plan is shown in Figure 35.

Masonboro Inlet, North Carolina

138. Masonboro Inlet is a natural inlet through the coastal beach of North Carolina located 8 miles northeast of Wilmington, NC. The inlet provided vessel passage from the ocean to the Atlantic Intracoastal Waterway and to various private and commercial docking facilities. Construction of a 3,679-ft-long jetty on the north side of the inlet with a 1,100-ft-long weir at a 0-ft el at its shoreward end and a deposition basin was completed in 1965. The plan functioned well until the entrance channel migrated toward the jetty and cut through the deposition basin, jeopardizing the structural

Figure 35. Surface current patterns for the optimum improvement plan at Little River Inlet, South Carolina

integrity of the jetty. To alleviate these problems, plans for the construction of an additional jetty south of the existing one were recommended.

139. A hydraulic model investigation was conducted to determine (a) the minimum length of the south jetty, (b) optimum alignment and spacing between the two jetties, (c) proper channel alignment, (d) shoaling and scouring characteristics, (e) influx of sediment into the entrance, and (f) current patterns at the entrance (Seabergh 1976). The model was equipped with necessary appurtenances to reproduce and measure tidal elevations, current velocities, waves, and sediments used in shoaling tests. Prior to testing of various improvement plans, the model was verified with prototype data.

140. The originally proposed improvement plans consisted of an 1,800-ft-long south jetty, and variations initially considered included lengthening the jetty and shifting it closer to the north jetty. Preliminary testing examined tidal surface current patterns for various structural configurations, including training structures to deflect currents away from the north jetty, offshore breakwaters, and various south jetty alignments. A

3,400-ft-long south jetty was determined to be the best plan tested. It was noted that the south jetty, about the same length of the north jetty, prevented a swing of flood currents toward the north structure, and flood flows showed good alignment at the entrance and through the region between the jetties (Figure 36), with the flow lines concentrated in the center of the jetties. With this plan in place, there should be no significant change to the tidal prism of the inlet, and bay elevations and velocities should remain very similar to existing conditions for the optimum plan. Tests indicated, also, that an increase in currents over the weir portion of the north jetty should occur, resulting in greater littoral movement to the deposition basin.

Figure 36. Optimum jetty configuration developed for Masonboro Inlet, North Carolina

Oregon Inlet, North Carolina

141. Oregon Inlet, the northernmost inlet through North Carolina's barrier islands, is a natural channel passing tidal flows between the Atlantic Ocean and extensive open bay sounds. Typical of many natural inlets, navigation through the inlet can be dangerous due to shallow shifting sand shoals, as evidenced by numerous occurrences of damage sustained by fishing vessels. The necessity of continued maintenance dredging and the exposure of commercial and private craft to shoaling and breaking waves indicated that inlet stabilization by jetties was desirable.

142. A hydraulic model investigation was conducted to determine (a) optimum jetty alignment and spacing between jetties, (b) minimum length of the

jetties, (c) navigability of the inlet with respect to eliminating adverse flow conditions, (d) if natural flow exchange between the ocean and bay would be maintained, (e) the stability of the channel, (f) effects of storm surge water levels on flow through the jettied inlet, (g) regions of scour and fill, and (h) wave heights in the system (Hollyfield, McCoy, and Seabergh 1983). The model was equipped with devices to generate and control tides, measure water-surface elevations, measure velocities, obtain surface currents, create storm surges, generate waves, and obtain photographs. Prior to testing improvements, the model was verified with prototype data.

143. The originally proposed improvement plans consisted of different jetty lengths and alignments with various spacings between them. A jetty alignment that was nearly parallel to the bar channel was selected as optimum (Figure 37). The jetties were 2,500 ft apart, and as a result of testing each jetty was decreased by 800 ft in length. This plan concentrated currents along the existing channel, reduced flood velocities thereby reducing the rate of sediment transport into the sound, and reduced velocities for ebb and flood storm surges. Model tests indicated that the jetty alignment would not negatively impact tidal exchange.

Newport News Harbor, Virginia

144. The Commonwealth of Virginia Department of Highways and Transportation proposed construction of a bridge/tunnel crossing at Hampton Roads between the cities of Newport News and Suffolk. The tunnel would pass below the existing Newport News navigation channel, and islands were needed on each end of the tunnel to provide a transition between the underwater tunnel and the surface approach structures. The construction of the north island tunnel would require reconstruction of the harbor entrance and relocation of the existing channel. Proposals were to relocate the harbor entrance about 150 ft eastward of its present location and increase the channel width from 60 to 90 ft to accommodate future improvements in the area.

145. A 1:75-scale model was designed and constructed to determine wave conditions in the harbor as a result of the proposed modifications to the entrance (Bottin 1984b). Waves with periods ranging from 3.5 to 6.0 sec and heights ranging from 1.7 to 9.0 ft were reproduced from five directions of wave approach with an swl of +2.6-ft mllw.

146. Model tests were conducted for existing conditions and 18 variations in the design elements of two basic jetty plans. One plan included a

Figure 37. Jetties aligned parallel to the bar channel were optimum for Oregon Inlet, North Carolina

rubble-mound jetty and one entailed a concrete pile jetty, and both were constructed in conjunction with the north tunnel island bayward of the existing entrance. Storm waves from various directions produced relatively calm wave conditions in the existing harbor with wave heights in excess of 1.0 ft only for the most severe storm waves (50-year recurrence).

147. The first harbor configuration entailed a 1,225-ft-long rubble-mound jetty with a +12.3-ft crest el. Tests indicated the structure could be reduced in length by 300 ft and the crest elevation could be reduced by 3 ft in height (Figure 38) with no adverse effects on wave conditions in the harbor. The second harbor configuration consisted of a 2,710-ft-long concrete cylinder-pile (66-in.-diam) breakwater with the piles spaced 6 in. apart and the openings sealed with timber from the crest to a -0.7-ft el. This plan resulted in excessive wave heights in the existing harbor. Test results

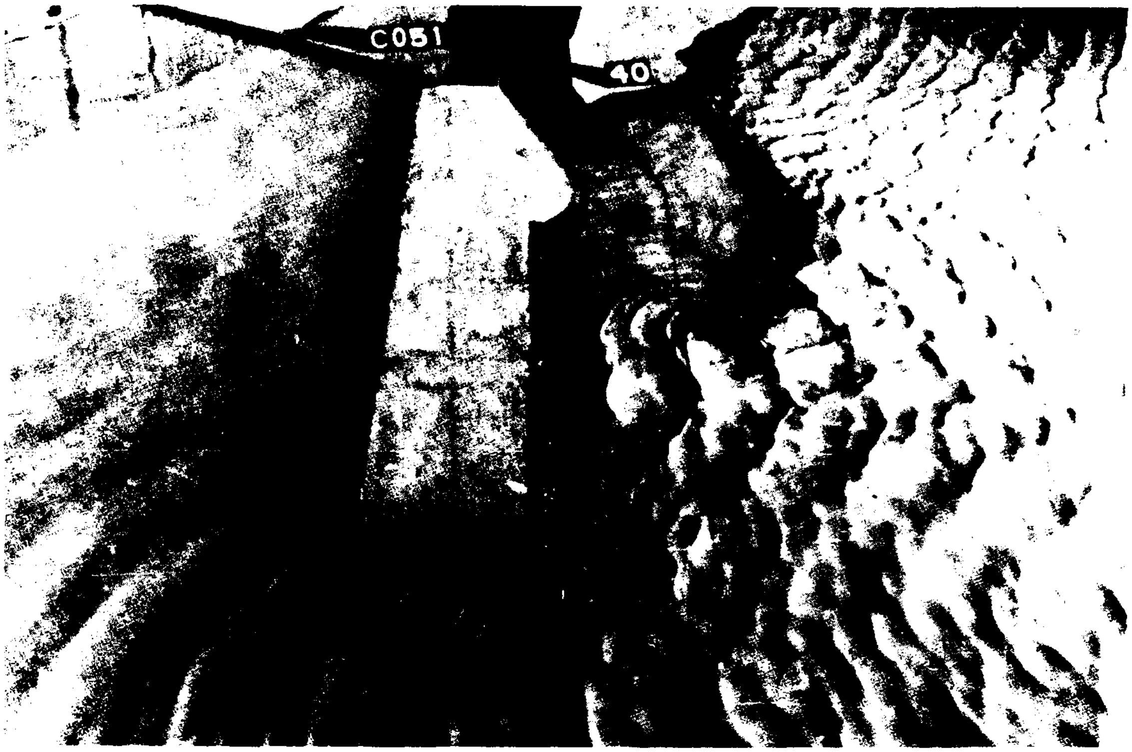

Figure 38. Optimum rubble-mound jetty configuration developed for Newport News Harbor, Virginia

indicated that the outer 1,035 ft of the structure could be sealed from the crest to a -4.7-ft el and result in no adverse effects on wave conditions in the existing harbor.

Barnegat Inlet, New Jersey

148. Barnegat Inlet is located on the New Jersey coast about 32 miles north of Atlantic City. It provides passage between the Atlantic Ocean and Barnegat Bay. Construction of converging north and south jetties was completed in 1940 to stabilize the inlet opening and provide protection to the entrance. After a period of time, both the outer and inner channels became extremely unstable in both depth and alignment. Sediment accumulated in the entrance, due to tidal currents and wave action, and maintenance dredging was very difficult due to unfavorable sea conditions. Plans for improving navigation conditions in the inlet were developed.

149. A hydraulic model investigation was conducted to evaluate the effectiveness of proposed plans of improvement for maintaining a stable navigation channel through the inlet (Sager and Hollyfield 1974). Tests were conducted to define the effects of each stage of a multistage improvement plan

on the hydraulic characteristics of the inlet. Prior to testing of various plans of improvement, comprehensive tests were conducted to verify the model with prototype data. The model was equipped with necessary appurtenances for generating tides and waves and for measuring tidal heights, velocities, and bottom elevations. In addition, photographic techniques were used to delineate surface current patterns.

150. The improvement plan developed for Barnegat Inlet entailed construction to be accomplished in seven stages. Model tests, however, indicated that two additional construction stages should be accomplished to even the distribution of flow and prevent crosscurrents that existed between the jetties. Analysis of model test results indicated that construction of a new south jetty was more effective for maintenance of a suitable navigation channel than raising the north jetty, and priority of construction should be given to the south jetty (Figure 39). In addition, it was determined that the parallel jetties would probably eliminate three of the construction stages bayward of the entrance. The channel dredged initially between the jetties was inadequate to preserve the tidal discharge and concentrate the flow along the alignment of the channel. The model study recommended new location, alignment, and dimensions of the entrance channel.

Shrewsbury Inlet, New York

151. Shrewsbury Inlet is located in New York Harbor south of Sandy Hook Bay. A small-boat channel was proposed across the base of Sandy Hook Peninsula for recreational boating and commercial navigation.

152. A 1:100-scale hydraulic model investigation was conducted to determine (a) the optimum location and length of the protection jetties, (b) transmission of wave energy through the inlet, and (c) detailed current velocities in critical locations for various flood and ebb discharges through the proposed small-boat inlet (McNair and Hill 1972). Waves with periods ranging from 6.6 to 11.2 sec and heights ranging from 2.7 to 10.5 ft were reproduced from one direction for swl's of 0.0-, +2.3-, and/or +3.9-ft mean low water (mlw). For some tests, flood and ebb tidal flows were reproduced through the entrance.

153. Seven improvement plans were tested which involved different channel alignments and depths and/or various lengths and locations of the jetties. The model revealed that 800-ft-long jetties spaced 508 ft apart with a channel extending straight through the jetties and then turning southwesterly to an

Figure 39. Construction of the new south jetty in Barnegat Inlet, New Jersey, was effective for maintenance of a suitable navigation channel

existing Federal channel were optimum. A view of waves entering the entrance is shown in Figure 40. Tests indicated that the optimum jetty configuration would result in current velocities in the new inlet that would not be excessive for safe navigation during normal tides. Wave energy originating in the ocean and passing through the new inlet would have insignificant effects on wave heights inside the bay, and the wave climate between the jetties should not be difficult to navigate except under extreme ocean conditions in combination with critical ebb discharges in the inlet.

Newburyport Harbor, Massachusetts

154. Newburyport Harbor is located on the northern coast of Massachusetts, about 54 miles north of Boston. North and south jetties were constructed at the harbor entrance in 1899 that were 4,118 and 2,445 ft long, respectively. During large storms, irreparable damage to the riverbank occurred inside the south jetty. Waves also overtopped the north jetty and

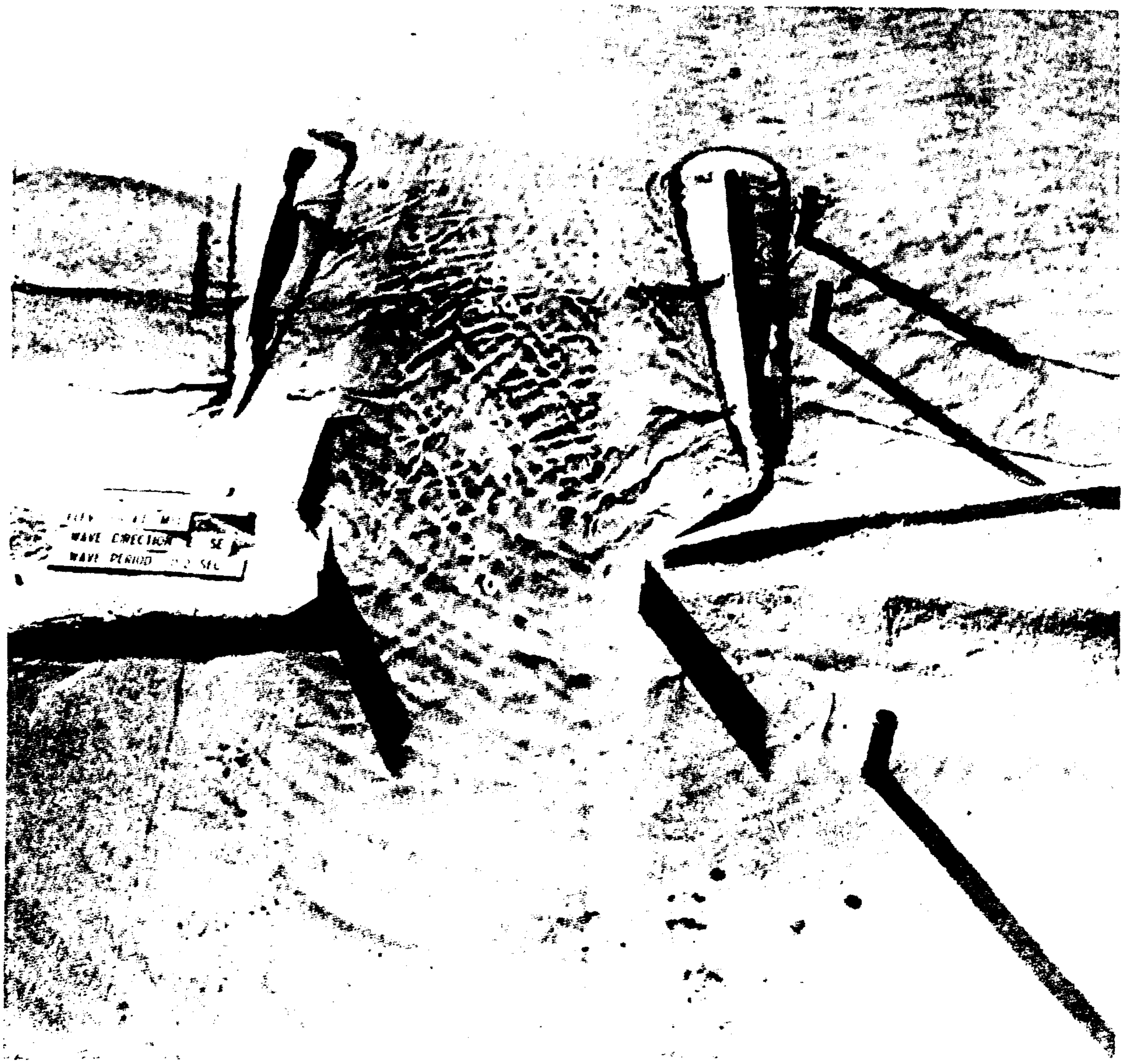

Figure 40. Jetty configuration tested in the Shrewsbury Inlet, New York, model

eroded sand in front of a Coast Guard station located there. A revetment was installed in front of the Coast Guard station and the erosion problem was subsequently transferred upriver.

155. A 1:75-scale hydraulic model study was conducted to study wave, shoaling, and erosion problems at the site and determine the effects of various improvement plans (Curren and Chatham 1979). Waves with periods ranging from 7 to 15 sec and heights ranging from 4 to 18 ft were reproduced from three deepwater directions with swl's of 0.0-, +2.9-, and/or +5.3-ft mean sea level (msl). Maximum flood and ebb tidal discharges also were reproduced

through the entrance in the model. These tidal discharges were combined with a typical freshwater discharge (8,200 cfs) and the resultant combined discharges were reproduced in the model.

156. Model tests were conducted for existing conditions and 13 test plan configurations which consisted of changes in the length and crest elevation of the north breakwater and the addition of groins at various locations. Test results for existing conditions revealed rough and turbulent wave conditions in the entrance channel due to waves breaking on a shoal across the entrance, waves diffracting around the jetty heads, and waves overtopping the north jetty. Tracer tests indicated that the model accurately reproduced the general sediment patterns observed in the prototype (as evidenced by visual observations and aerial photographs).

157. Initially, improvement plans were tested that entailed raising the crest elevation of the north breakwater from +8.3 ft (existing elevation) to elevations ranging from +11 to +17 ft. North breakwater extensions to 1,000 ft in length with +8.3 and +11 ft crest el then were tested. In addition, north and south groins were installed inside the entrance with a +10-ft el. Model tests revealed that an 850-ft-long south groin in conjunction with raising the existing north jetty to an +11-ft el (Figure 41) would provide adequate erosion protection while improving entrance wave conditions. This plan was considered optimum with regard to the protection provided and construction costs.

Wells Harbor, Maine

158. Wells Harbor, Maine, is a small inlet located at the mouth of the Webhannet River about 20 miles northeast of Portsmouth Harbor, Maine. It is primarily a summer resort area for small pleasure boats. Construction of an 840-ft-long north jetty and a 940-ft-long south jetty was completed at the site in 1962. Excessive shoaling occurred in the harbor entrance after jetty construction, and in 1967 the north and south jetties were extended by 1,225 and 1,300 ft, respectively. These jetty extensions were unsuccessful in eliminating shoaling problems. The entrance channel did not maintain a self-scouring depth and frequent dredging was required. In addition, little wave protection was afforded to small boats navigating the channel during periods of storm wave activity.

159. A 1:50-scale hydraulic model was designed and constructed to study wave and current conditions in the Wells Harbor entrance channel both with and

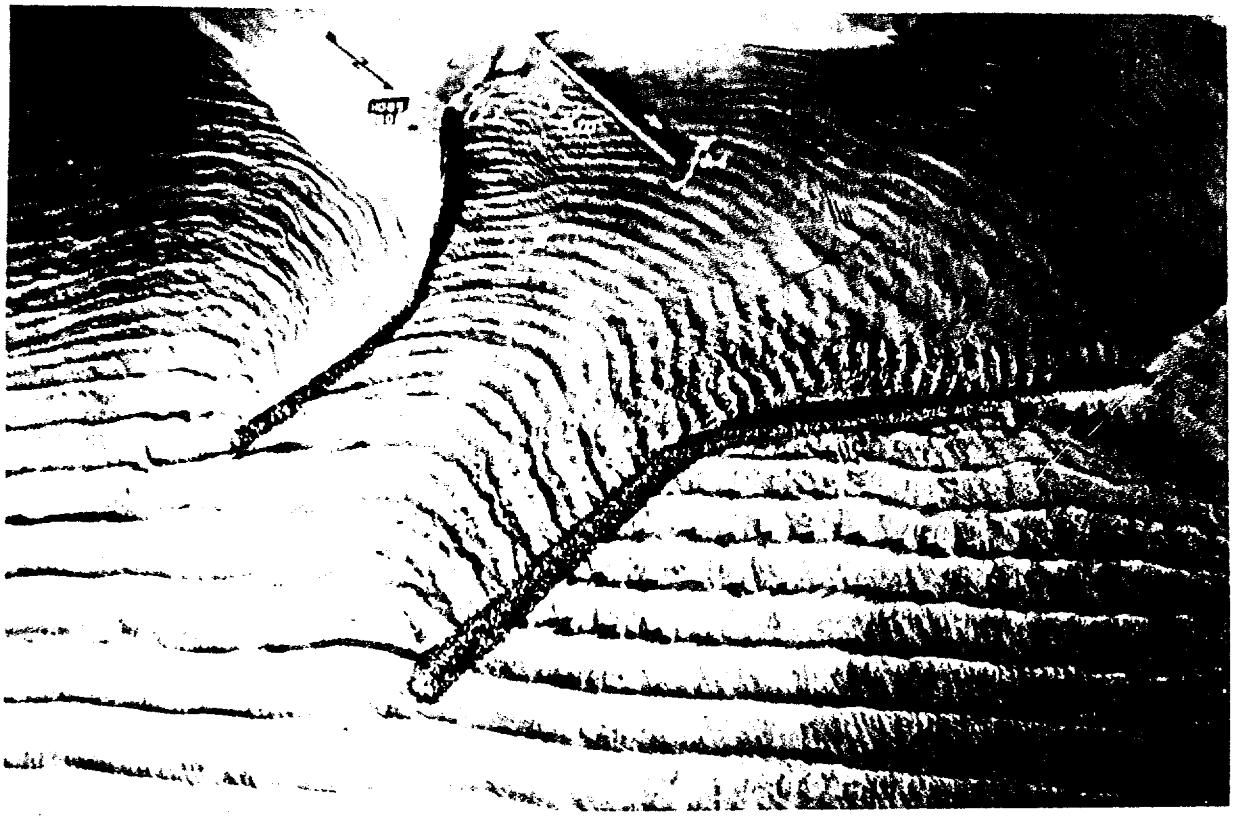

Figure 41. Optimum plan of improvement for Newburyport Habor, Massachusetts

without the proposed improvements installed (Bottin 1978). Waves with periods ranging from 5 to 17 sec and heights ranging from 4 to 14 ft were reproduced from one test direction using swl's of +4.5-, +6.8-, and/or +8.6-ft mlw. Maximum flood and ebb tidal flow conditions through the entrance also were reproduced in the model.

160. Model tests were conducted for existing conditions and three test plan configurations. Proposed improvements consisted of the installation of stone spur dikes in the jettied entrance and a breakwater attached to the existing north jetty. Test results for existing conditions indicated, in general, poor navigation conditions when waves were moderate to large. Waves break in the entrance for some conditions, and the problem was compounded for ebb tidal currents which tended to steepen and change the direction of incident waves.

161. The original plan configuration installed in the model consisted of 10 spur dikes that reduced the controlling width between jetties from 400 to 280 ft. Tests indicated that this plan increased wave heights in the outer

entrance. Changing cross sections of the dikes did not improve conditions at that location, but removal of selected spur dikes should not compromise design effectiveness. The installation of a breakwater (Figure 42) was the most effective plan tested and substantially improved wave conditions throughout the jettied entrance. Qualitative indications, with regard to the ability of the spur dikes to maintain a self-scouring channel, were that the dikes should be beneficial in reducing maintenance dredging requirements.

Port Ontario Harbor, New York

162. Port Ontario Harbor is located at the eastern end of Lake Ontario, at the mouth of the Salmon River, about 20 miles northeast of Oswego, NY. The area is principally recreational and agricultural, although there are several small manufacturing establishments upstream. A sand and cobble bar at the mouth of the river is frequently formed due to wave action. Because of the shallow depths and constant shifting of the bar across the entrance, numerous navigational difficulties were experienced. The entrance channel was virtually closed at the peak of the navigation season, when lake levels were normally low.

163. A 1:75-scale hydraulic model was designed and constructed to study shoaling, wave action, and riverflow conditions at the harbor entrance and lower reaches of the river both with and without proposed improvements (Bottin 1977b). Waves were reproduced with periods ranging from 5.2 to 11.3 sec and heights from 2.0 to 21.4 ft from three deepwater directions for +2.0- and/or +4.7-ft swl's low water datum (lwd). River discharges, up to 15,000 cfs were reproduced in the model.

164. Model tests were conducted for existing conditions including 11 test plan configurations with variations consisting of changes in the lengths, alignments, and crest elevations of the proposed breakwaters and the alignment of the entrance channel. For existing conditions, rough and hazardous wave conditions were measured in the river entrance during periods of storm wave attack. In addition, the model indicated that wave action will form a shoal across the river mouth which would interfere with navigation and the passage of riverflows.

165. The originally proposed plan of improvement entailed a 1,450-ft-long south jetty and a 360-ft-long north jetty. This plan met the sponsor's established 2-ft maximum wave height criterion at the creek mouth and had little effect on water-surface elevations in the lower reaches of the creek,

Figure 42. Most effective plan tested in the Wells Harbor, Maine, model

but it was not optimum considering prevention of shoaling. After additional tests, it was determined that a 460-ft-long north jetty and a 1,450-ft-long south structure were optimum with respect to shoaling protection. Model test results indicated that the south breakwater could be reduced in crest elevation by 2 ft (+12 to +10 ft) and not increase wave heights between the structures. The optimum improvement plan tested for Port Ontario Harbor is shown in Figure 43.

Figure 43. Optimum improvement plan for Port Ontario Harbor, New York

Oswego Harbor, New York

166. Oswego Harbor is located on the southern shore of Lake Ontario, about 15 miles from its easterly end, at the mouth of the Oswego River. The harbor was afforded some protection from storm waves by converging rubble-mound breakwaters which form a 650-ft-wide navigation opening. Storms, however, from north-northwest to north-northeast propagated through the entrance and caused considerable damage to harbor facilities. Plans were developed to protect the navigation opening and improve wave conditions in the harbor.

167. A 1:100-scale hydraulic model study was conducted to determine if a proposed breakwater was adequate to protect the harbor from storm waves, and if not, to devise a plan which would afford sufficient protection (Fortson

et al. 1949). Waves with periods ranging from 5.2 to 8.2 sec and heights ranging from 7 to 18 ft were reproduced from four deepwater directions of approach with +3.5-ft low water datum (lwd).

168. Tests were conducted for existing conditions and eight breakwater improvement plans. Existing conditions indicated that the most critical direction of wave approach, with regard to wave action in the harbor, was from north (Figure 44). Wave heights in excess of 10 ft were measured in the harbor. The originally proposed improvement plan consisted of removal of 1,030 ft of the shoreward end of the east breakwater with a 4,900-ft-long extension of this structure extending easterly to shore. A 1,000-ft-long detached breakwater was included lakeward of the entrance. Test results indicated that this plan would not afford adequate wave protection. The detached breakwater reflected westerly waves into the harbor. Although numerous plans were tested, the optimum breakwater alignment with respect to wave protection and cost entailed moving the detached breakwater 400 ft shoreward and decreasing its length by 350 ft. A spending beach in the corner of one of the basins also was necessary. Wave heights were reduced to 2 ft or less in the harbor for this plan.

Hamlin Beach Harbor, New York

169. Hamlin Beach Harbor was proposed for construction on the south shore of Lake Ontario, in Hamlin Beach State Park, which is about 17 miles northwest of Rochester, NY. There was no development at the site, and proposed improvements consisted of breakwaters to protect the harbor entrance, an entrance channel, and an interior channel adjacent to a berthing area. Storm waves in this area of the lake ranged up to 10 ft in height and made navigation difficult and dangerous for small craft near shore and caused serious damage to moored small boats.

170. A 1:64-scale hydraulic model was designed and constructed to determine the optimum length of the protective structures proposed at the harbor entrance with respect to economics and wave heights in the basin (Brasfeild 1973). Waves were reproduced from three directions of approach that ranged from 7 to 8 sec in period and 7.6 to 10.2 ft in height for a +5.0-ft swl. For a plan to be acceptable, the sponsor specified that wave heights were not to exceed 2.5 ft and 0.5 ft in the entrance and berthing area, respectively.

171. The originally proposed improvements consisted of the installation of a 950-ft-long west breakwater and a 270-ft-long east breakwater. Test

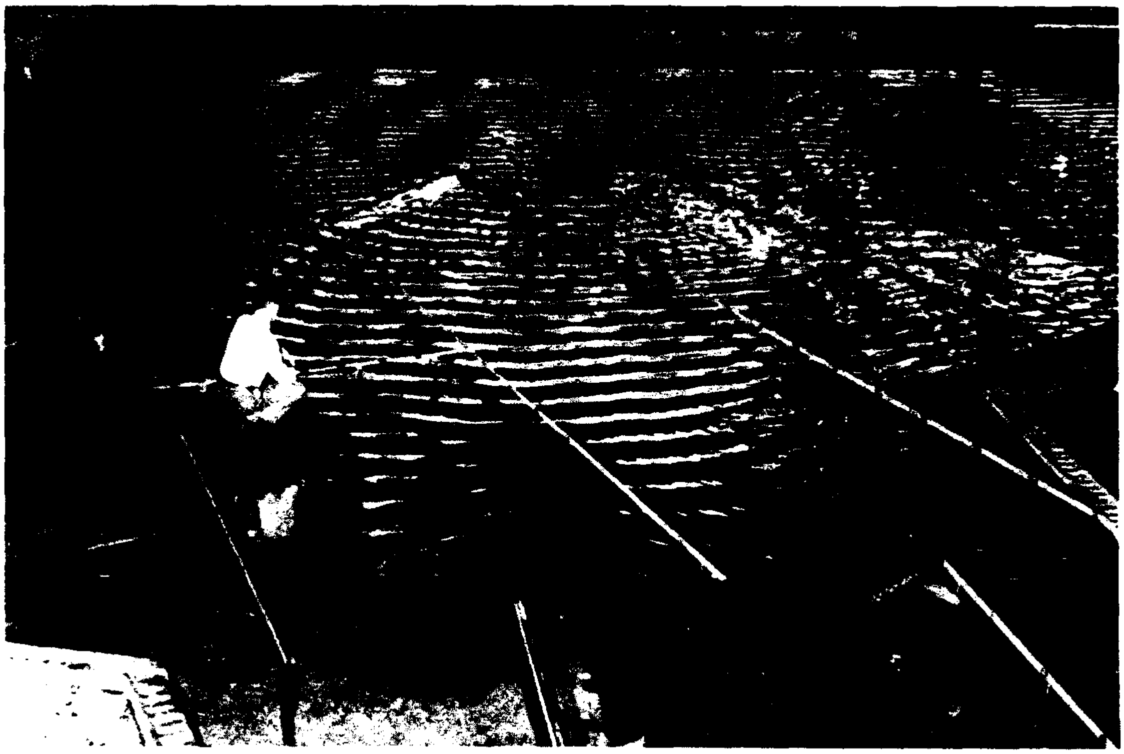

Figure 44. View of wave action from north entering the existing Oswego Harbor, New York

results revealed that wave heights were well within the established criterion, and that 100 ft of the lakeward end of west breakwater could be removed (Figure 45) and still provide adequate protection to the entrance and harbor. Test results also revealed that an additional 100-ft reduction in length of the west breakwater would not seriously impair the protection desired for the harbor. Wave height criteria should be exceeded slightly during storm wave attacks from the easterly directions.

Olcott Harbor, New York

172. Olcott Harbor, New York, is located on the southern shore of Lake Ontario at the mouth of Eighteenmile Creek, situated about 18 miles east of the mouth of the Niagara River. Concrete-capped, steel sheet pile, vertical, parallel jetties are located 200 ft apart at the creek mouth. The harbor was fully developed with boat docks and facilities on both banks of the creek and had a mooring capacity of 134 vessels. Waves in the entrance were extremely hazardous during storms when waves reflected off and overtopped the vertical jetties in the entrance. In addition, waves entering the harbor between the jetties broke vessels loose from their moorings resulting in vessel and

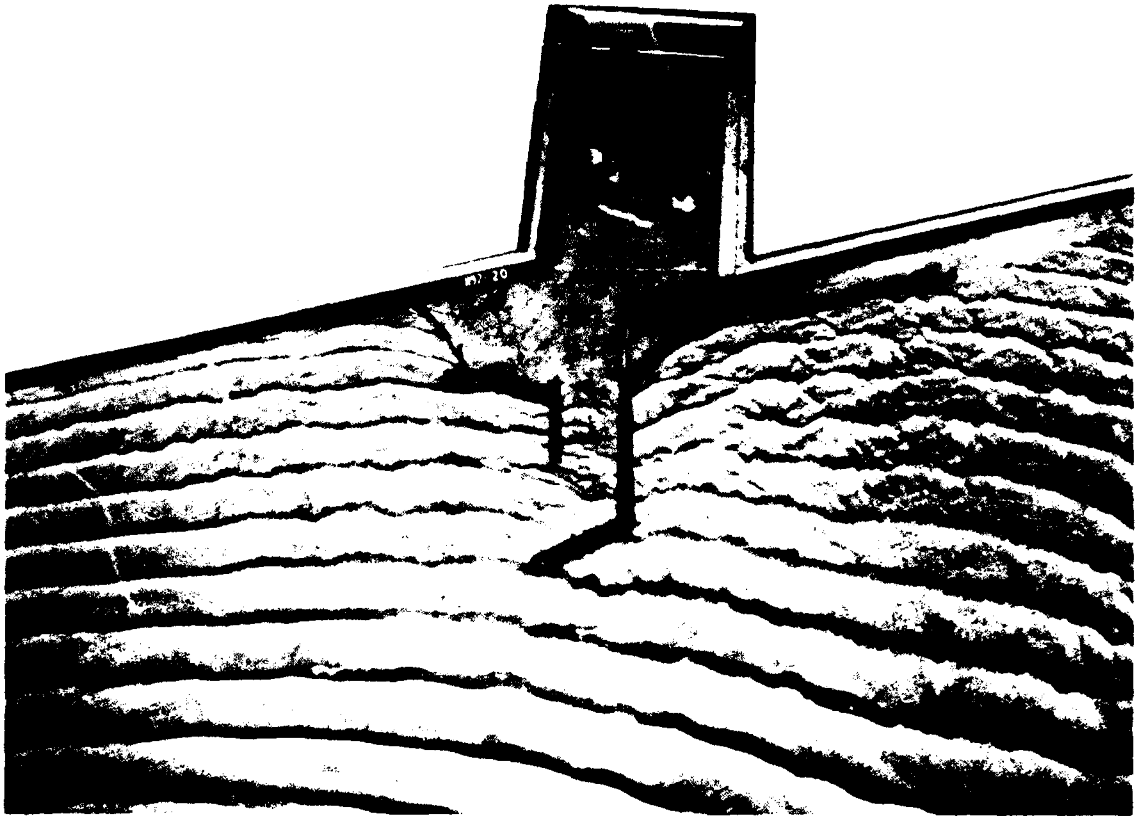

Figure 45. Optimum improvement plan developed for Hamlin Beach Harbor, New York

facility damage. The development of additional moorings in the creek was restricted, and an analysis of boating needs indicated an immediate need for more than 500 additional permanent berths. Development geared to providing both protection to the existing harbor and expansion of the harbor into Lake Ontario adjacent to the parallel jetties was proposed.

173. A 1:60-scale hydraulic model investigation was conducted to study wave, current, creek flow, and shoaling conditions at the harbor and determine if proposed improvements would provide adequate protection from these events (Bottin and Acuff 1990). Waves with periods ranging from 5.7 to 7.4 sec and heights ranging from 4 to 11 ft were reproduced from five directions by a spectral wave generator using swl's of +2.8- and/or +4.0-ft lwd. Creek discharges ranging from 1,500 to 5,100 cfs also were reproduced to determine water surface elevations and current velocities in the creek. For improvement plans to be acceptable, the sponsor specified that wave heights were not to

exceed 3 ft in the proposed entrance or 1 ft in the proposed mooring areas for wave conditions occurring during boating season.

174. Model tests were conducted for the existing harbor configuration and for 23 test plan variations of two basic harbor configurations. One configuration provided a mooring area to the east of the existing entrance, and one configuration provided mooring areas on both the east and west sides of the existing entrance. Test results for existing conditions revealed rough wave conditions in the entrance with heights up to 6.5 ft. Confused wave patterns were observed between the jetties due to reflections from the vertical-wall structures.

175. The first basic harbor configuration consisted of a 1,110-ft-long west dogleg breakwater, a 1,650-ft-long detached east breakwater, a 340-ft-long east spur breakwater, and channel dredging. A mooring area was provided on the east side of the existing entrance. The second basic harbor configuration provided for mooring areas on both the east and west sides of the existing entrance. It consisted of detached 1,570-ft-long west dogleg breakwater, a 270-ft-long west spur breakwater, a 1,650-ft-long detached east breakwater, and a 340-ft-long east spur breakwater.

176. Test results for both the first and second harbor configurations revealed that wave heights were well within the established criteria for both plans. Model tests indicated that the crest elevations of both breakwaters could be reduced in height and that the east breakwater could be reduced by 125 ft in length for both configurations. In addition, 350 ft could be removed from the west breakwater of the first harbor configuration and 50 ft could be removed from the west breakwater of the second harbor configuration and the established 1-ft maximum wave height criterion in the harbor could be met. Tests also revealed that the construction of either harbor plan would have minimal impact on water surface elevations and creek current velocities in the lower reaches of Eighteenmile Creek. A view of the second basic harbor configuration with optimum improvements is shown in Figure 46. The openings between the attached and detached breakwaters will provide wave-induced current flow through the harbor, however, the opening between the attached and detached west breakwaters of the second basic harbor configuration may result in minor shoaling in the mooring area in the western portion of the harbor for waves from westerly directions. The installation of a sill between the

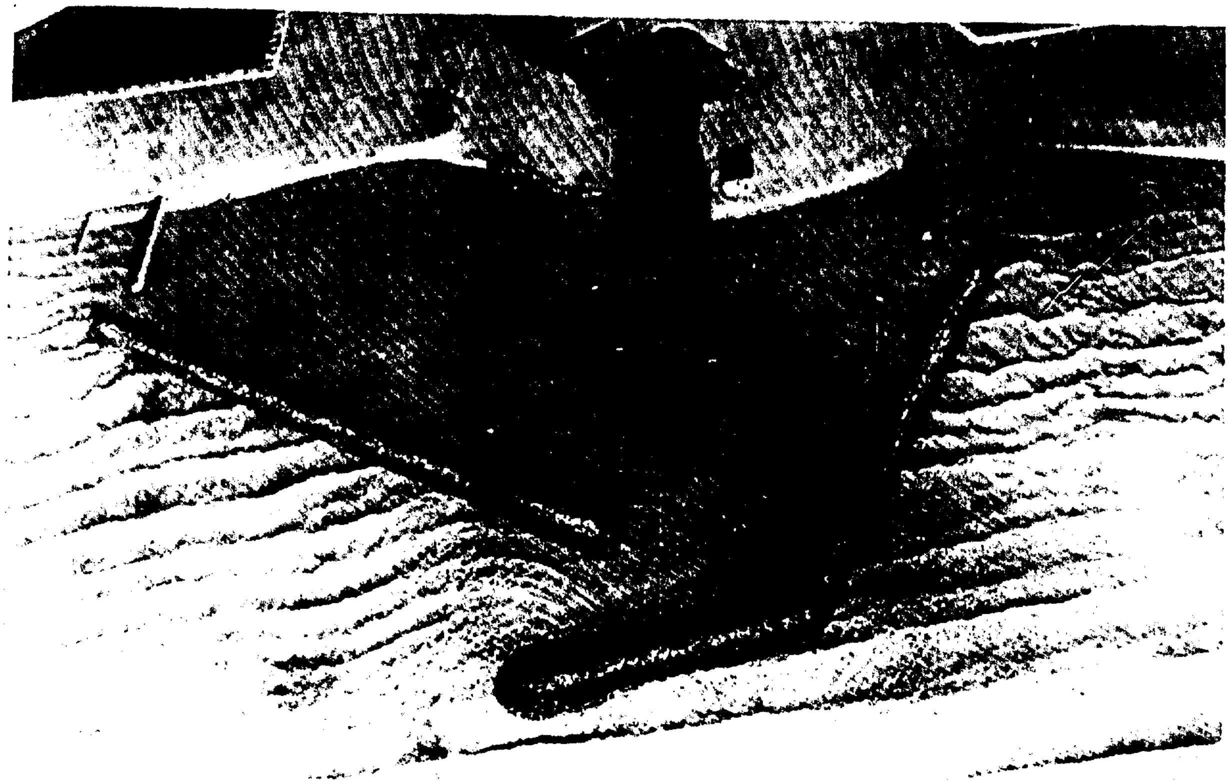

Figure 46. Optimum plan of improvement for the second basic harbor configuration tested in the Olcott Harbor, New York, model

structures, an extension of the shore-connected breakwater, or a spur on the shoreconnected structure should alleviate this shoaling.

177. Additional tests were conducted for the second basic harbor configuration to determine the effects of the proposed improvements on creek temperatures and currents at Eighteenmile Creek as they entered the Lake (Bottin 1990). Tests were conducted to verify the performance of the model with prototype data obtained for existing conditions. A reservoir of heated water was used to introduce discharges into the creek for the spring and fall seasons. The discharge water was dyed so that current patterns could be traced. It was concluded that the proposed improvements should have no adverse impact on temperature variations or the movement of creek water into the lake and along the shoreline. Similar trends with regard to temperature variations were measured in the entrance and the lake, and movement of the creek plume into the lake and along the shoreline varied slightly in a localized area at the entrance but was similar on a regional basis for the improvement plan.

Cattaraugus Creek Harbor, New York

178. Cattaraugus Creek enters the lake on the south shore of Lake Erie about 24 miles southwest of Buffalo, NY. The harbor consisted of the lower 3/4 mile of the creek where over 400 small boats were permanently based. The economy of the immediate area was primarily recreational. Flooding occurred almost every year along the lower reaches of the creek primarily due to the presence of a restrictive sand and gravel bar at the creek mouth. The bar, formed by littoral drift due to wave action, at times virtually closed the outlet and caused navigational difficulties because of shallow depths and the constant shifting of the bar across the entrance. A plan of improvement was proposed to provide wave protection to the harbor and prevent the formation of the sand and gravel bar at the creek mouth.

179. A 1:75-scale hydraulic model investigation was conducted to study shoaling, wave action, and flood and ice flow conditions at the harbor entrance and lower reaches of the creek both with and without proposed improvements (Bottin and Chatham 1975). Waves with periods ranging from 6 to 9 sec and heights ranging from 4 to 14 ft were reproduced from three directions of approach for swl's of 0.0- and/or +3.0-ft lwd. Creek discharges ranging from 10,000 to 57,900 cfs were reproduced in the model. Crushed coal was used to simulate sediment in the model and a low-density polyethylene sheet material was used to simulate ice.

180. Model tests were conducted for existing conditions and nine test plan configurations. For existing conditions, the model indicated that wave action would form a shoal across the creek mouth which would seriously interfere with navigation and the passage of flood flows and ice. During periods when the shoal was absent (washed out by flood flows) wave heights would be excessive in the creek mouth and lower reaches of the creek. The originally proposed improvement plan consisted of a navigation opening and entrance channel (protected from waves by a sheet-pile breakwater) oriented toward the west. This plan resulted in excessive shoaling of the entrance. Since the predominant direction of littoral drift at and near the mouth would shoal the entrance, the breakwater orientation was not considered feasible.

181. Additional testing indicated that a navigation opening oriented toward the east with rubble-mound breakwaters aggregating 2,450 ft was the optimum plan (Figure 47). The plan showed no ice jamming tendencies and should help prevent the formation of windrowed lake ice at the entrance. Wave

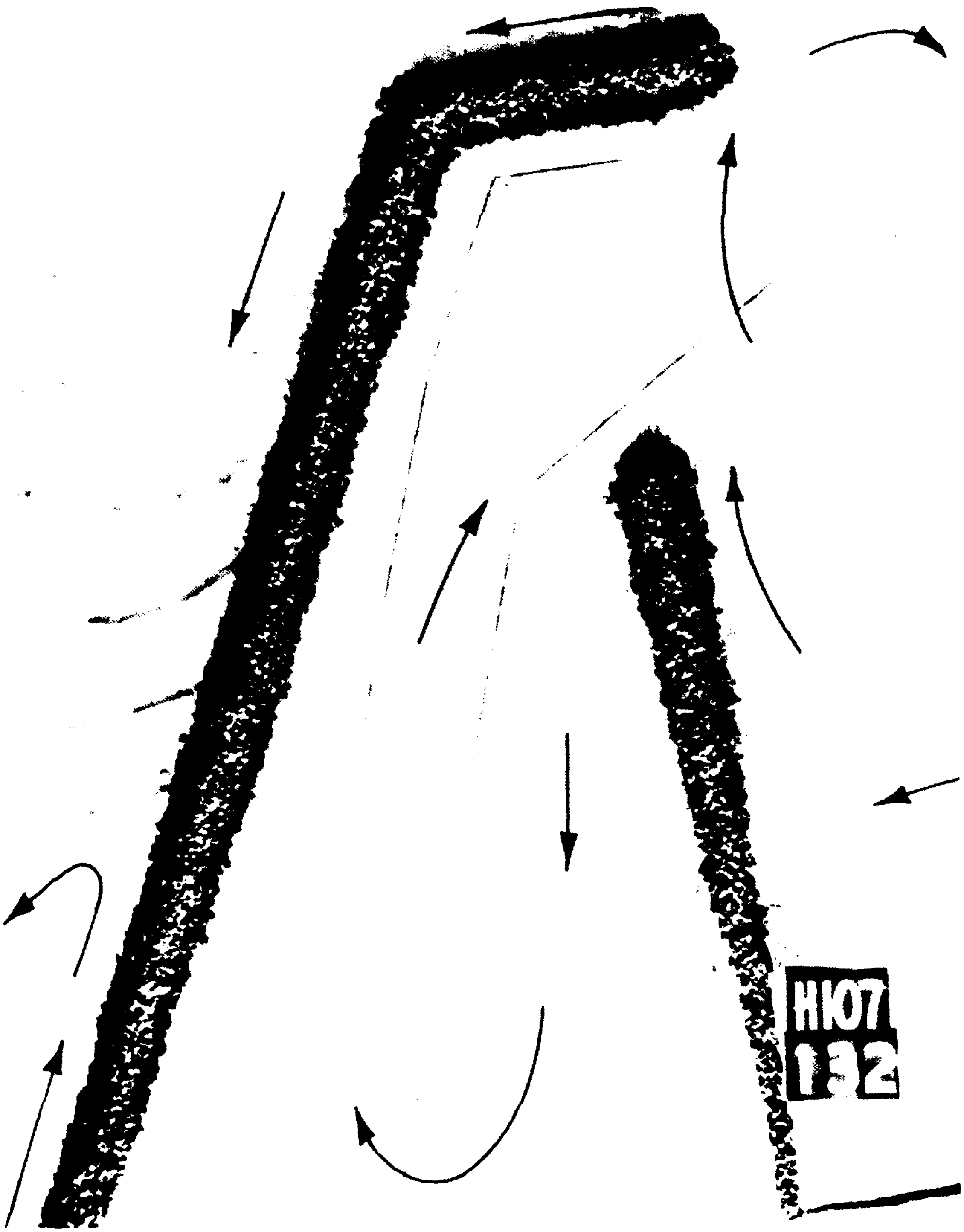

Figure 47. Optimum breakwater configuration developed in the model study of Cattaraugus Creek, New York

heights in the entrance and lower reaches of the creek were acceptable. A vertical wall, steel sheet-pile structure was tested which was on the same alignment as the optimum rubble-mound breakwaters. The rubble-mound structure proved to be more satisfactory for the passage of flood flows because some flow can escape through the voids of the breakwater. Also, wave energy reflecting off the vertical walls of the sheet-pile breakwaters may adversely affect small boats using the entrance and may stimulate erosion in the vicinity of the structures.

Barcelona Harbor, New York

182. Barcelona, located on the southern shore of Lake Erie about 60 miles west of Buffalo, was the site of a proposed harbor. It was exposed to wind waves reproduced by storms from all directions from west-southwest clockwise to northeast. There was a shallow-water mooring area which was protected from westerly storm waves by a short peninsula aligned in a north-south direction. However, a dredged harbor with protecting breakwaters was desired to provide protection for recreational, light-draft fishing, and other vessels for waves from all storm directions.

183. A 1:68-scale hydraulic model study was conducted to determine if a proposed arrowhead-type breakwater system would provide adequate wave protection for small craft anchored in the enclosed mooring action (Jackson, Hudson, and Housley 1959). Waves with periods ranging from 4 to 6 sec and heights ranging from 3.2 to 8.6 ft were reproduced from five directions of approach for a +3.7-ft swl lwd.

184. Model tests were conducted for existing conditions, the proposed plan, and six modifications of the proposed plan. For a plan to be acceptable to the sponsor, wave heights were not to exceed 2 ft in the proposed mooring area. Test results obtained for existing conditions indicated wave heights of 4 ft in the mooring area for the selected test conditions. The proposed improvement plan consisted of a 794-ft-long west breakwater and an 890-ft-long east breakwater forming a 200-ft-wide entrance channel. The structures were concrete-capped cellular sheet pile. Tests indicated wave heights up to 3 ft in the mooring area for this plan.

185. Through model testing, the study revealed that the width of the navigation opening should be reduced from 200 ft to 150 ft, an angle in the west breakwater should be eliminated, and that 200 ft could be removed from the shoreward end of the east breakwater.

186. The breakwaters were subsequently constructed in the prototype, however, a vertical faced public wharf was later constructed. Waves propagating into the harbor and reflecting off the wharf and vertical cellular breakwaters resulted in a confused wave climate inside the harbor with standing and multidirectional waves. Wave heights of 3 to 4 ft were not uncommon in the harbor.

187. Another model investigation was conducted at a 1:60-scale to develop plans that would provide adequate wave protection (Bottin 1984a). Waves with periods ranging from 5.7 to 10.1 sec and heights ranging from 2.9 to 14.7 ft were reproduced from four directions for swl's of +3.0-, +4.0-, +5.5-, and/or +6.5-ft lwd, depending on the season of the year. Note that test conditions were significantly higher in this study as compared to the one conducted 25 years earlier.

188. Tests were conducted for the existing harbor configuration and 58 test plan variations that consisted of changes in the lengths, alignments, and cross sections of lakeward breakwater extensions; shoreward extensions of the east breakwater; absorbers on the harbor sides of the breakwaters; the installation of a parapet wall on the west breakwater; and an absorber along the vertical faced city dock. Test results for the existing configuration revealed very confused wave patterns in the harbor with heights exceeding 4.0 ft in the mooring area. Testing of the originally proposed improvement plans indicated that absorbers inside the harbor and shoreward extensions of the east breakwater would not reduce wave heights to acceptable levels. Results revealed substantial wave energy entering through the arrowhead entrance.

189. The model study revealed that a 250-ft-long lakeward extension of the west breakwater, with a 790-ft-long absorber along the existing west breakwater, in conjunction with a 150-ft-long shoreward extension of the east breakwater, was required to provide adequate wave protection. Additional testing revealed that four 100-ft-long sections of the west breakwater absorber could be removed (Figure 48) without significant impact on wave heights in the mooring areas. Also, if the vertical-wall city dock was removed from the harbor, the 150-ft-long shoreward extension of the east breakwater could be removed without sacrificing wave protection in the mooring area.

Figure 48. Optimum improvement plan developed in the 1:60-scale model of Barcelona Harbor, New York

Conneaut Harbor, Ohio

190. Conneaut Harbor is located on the south shore of Lake Erie at the mouth of the Conneaut River about 75 miles northeast of Cleveland, OH. The harbor consisted of a triangular-shape, outer harbor protected by two converging breakwaters, and a rectangular-shape, inner harbor, which was formed by the lower portion of the river. East and west piers provided a 200-ft-wide entrance to the inner harbor. The breakwater system did not provide protection to the inner harbor for storm waves from the northwesterly directions. Navigational difficulties were experienced at the mouth of the inner harbor due to wave and current conditions.

191. A 1:125-scale hydraulic model investigation was conducted to determine improvements that would provide a satisfactory reduction in currents across the entrance to the inner harbor and reduction of wave conditions in the inner harbor, as well as protection of the outer navigation entrance (Hudson and Wilson 1963). Waves with periods ranging from 5 to 7 sec and heights ranging from 5 to 12 ft were reproduced from seven directions of wave

approach for a +2.8-ft swl lwd. A seiche current of 2 fps also was reproduced in the model.

192. Model tests were conducted for existing conditions and 21 test plan configurations. Results for existing conditions revealed wave heights up to 5 ft in the inner entrance and 3 ft in the inner harbor. Crosscurrents of 1.1 fps were measured across the entrance to the inner harbor.

193. At the conclusion of the model testing it was recommended that (a) the east breakwater be extended to shore, and (b) the navigation entrance into the inner harbor be increased in width (by removal of the entire east pier, or a portion of it with a realignment of the remaining portion parallel to the west pier). To provide the desired degree of wave reduction in the inner harbor, construction of a 900-ft-long detached breakwater (Figure 49) also was recommended.

Geneva-on-the-Lake Small-Boat Harbor, Ohio

194. Geneva-on-the-Lake is located on the south shore of Lake Erie about 17 miles east of Fairport, OH. Bordering the shoreline is the recreational development of Geneva State Park. The Park offered no harbor facilities for boaters desiring to use Lake Erie, so a project was proposed which would provide for a small boat harbor-of-refuge and recreational fishing facilities.

195. A 1:60-scale hydraulic model investigation was conducted to determine the most economical breakwater configuration that would provide adequate wave protection for small craft in the harbor (Bottin 1982a). Waves with periods ranging from 5.5 to 9 sec and heights ranging from 4.4 to 10.9 ft were reproduced from three wave directions with swl's of +0.3-, +0.9-, +4.0-, +4.4-, and/or +5.3-ft lwd. Discharges of 65 and 800 cfs from an adjacent wetland area also were reproduced in the model.

196. Wave height tests were conducted for 29 test plan variations of the basic harbor design. Variations consisted of changes in the lengths, alignments, crest elevations and/or cross sections of the breakwater structures. For a plan to be acceptable to the sponsor, wave heights were not to exceed 4.0 ft in the entrance channel and 1.0 ft in the mooring area. The original improvement plan consisted of a 400-ft-long rubble-mound east breakwater (crest el +6.6 ft) and a 650-ft-long rubble-mound west breakwater (crest el +5.9 ft) positioned in an arrowhead configuration. Revetments extended along the entire western side of the harbor on a levee adjacent to the

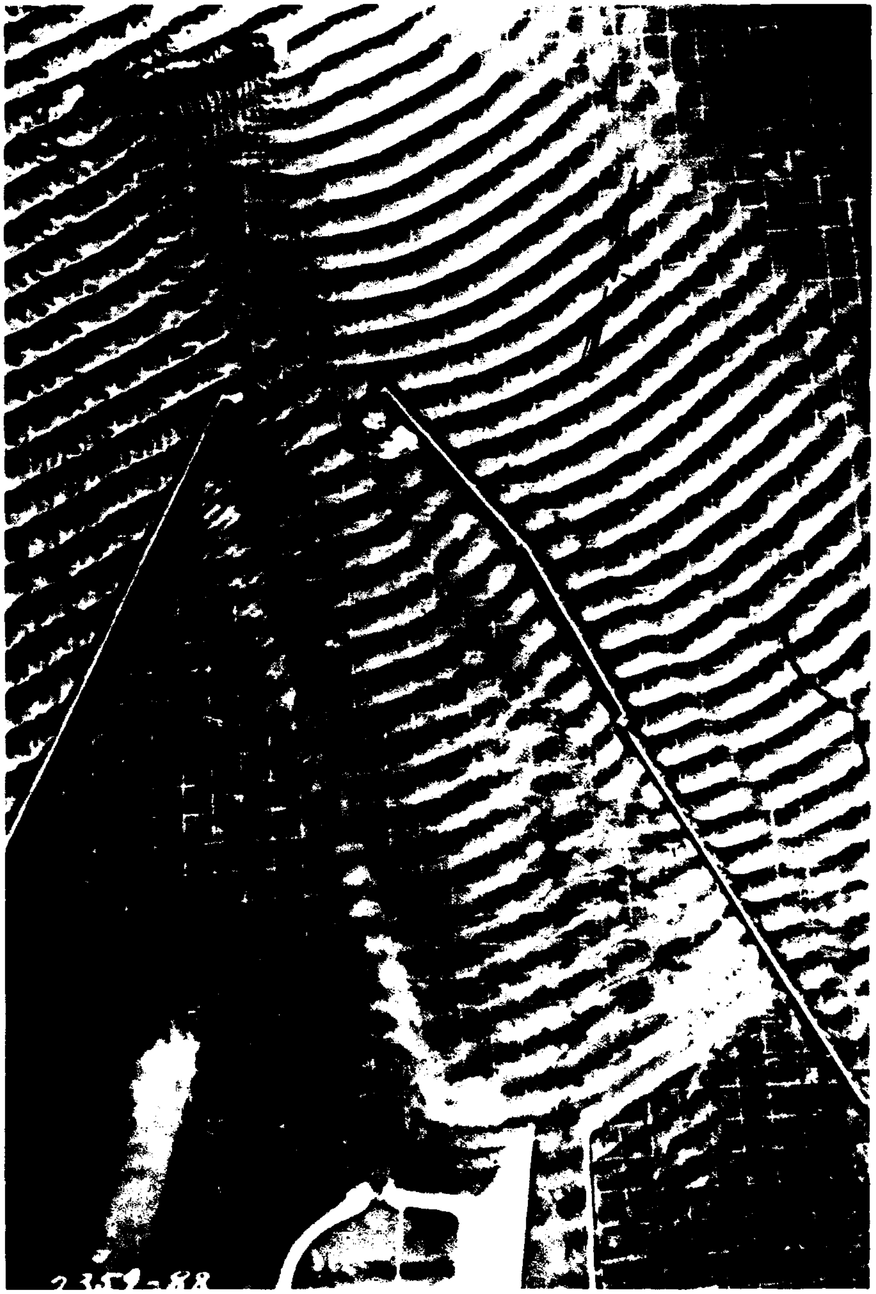

Figure 49. Optimum improvement plan developed for Conneaut Harbor, Ohio

wetlands and also on the eastern side of the entrance for about 200 ft. Test results indicated that the wave height criterion would be exceeded in the entrance channel and mooring areas due to significant overtopping of the breakwaters.

197. Numerous improvements were tested before an optimum plan was selected. This plan entailed raising the proposed east and west breakwaters to +8-ft el and installing 200-ft extensions to the east and west structures parallel to the entrance channel. Small spurs were needed at the revetted entrance which extended lakeward about 25 ft with +6-ft crest el. This plan (Figure 50) met the desired wave height criteria in the harbor, and tracer tests indicated that sediment would not move into the small-boat harbor entrance. A remote-controlled model cabin cruiser used in conjunction with the hydraulic model qualitatively indicated no significant problem in the harbor with regard to boat-generated standing waves. The initial wake from the boat did reflect off the walls but tended to dissipate quickly.

Figure 50. Optimum improvement plan for Geneva-on-the-Lake Small-Boat Harbor, Ohio

Chagrin River, Ohio

198. The Chagrin River is located in northeastern Ohio and flows into Lake Erie at Eastlake. The lower 1.5 miles of the river was extensively developed for mooring of small boats. The mouth of the river, however, had many times been virtually closed by sandbars formed by river currents and littoral drift due to wave action. Formation of the entrance sandbar contributed to

flooding upstream and was restrictive to passage of ice. In addition, difficulty had been experienced in maintaining a navigable channel for small boats. A plan of improvement was proposed to provide wave protection and prevent formation of the sandbar in the river mouth.

199. A 1:75-scale hydraulic model was designed and constructed to study wave action and flood flow conditions in the harbor entrance and lower reaches of the river with proposed improvements installed (Chatham 1970). Waves with periods ranging from 5 to 9 sec and heights ranging from 5 to 11 ft were reproduced from seven directions with swl's of +1.4- and/or +2.8-ft lwd. River discharges were reproduced for river flows of 20,000 and 27,000 cfs.

200. Tests were conducted for 18 variations in design elements of the proposed improvement plan. The originally proposed improvement plan consisted of arrowhead breakwaters aggregating 2,360 ft in length with a 275-ft navigation opening between the breakwaters and a 230-ft-wide entrance channel. Wave heights were not to exceed 2.5 ft in the main channel and 1.5 ft in the east channel and mooring basin entrances. Results of wave height tests for the originally proposed improvement plan revealed wave heights of 3.2 ft in the channel, 2.7 ft in the entrance to the east channel, and 1.7 ft in the entrances to the boat mooring basins. Current measurements indicated the originally proposed plan would result in velocities that would exceed the select criteria.

201. After numerous tests, an optimum plan was selected which entailed increasing the length of the east breakwater by 90 ft, thereby reducing the navigation opening between jetties to 190 ft. A 330-ft-long rubble-mound wave absorber was installed between the east and main channels with a 150-ft-long rubble-mound groin (Figure 51). This plan would meet the specified criteria with respect to wave protection at all locations in the harbor and the selected criteria with respect to river current velocities and the passage of flood flows. It was concluded that installation of a wave absorber on the vertical cellular sheet-pile east breakwater along the lakeward face would effectively reduce heights of reflected waves from the structure. The installation of groins along the shore east of the east breakwater would cause formation of eddies in wave-induced current patterns alongshore and should help to reduce erosion.

Edgewater Marina, Ohio

202. Edgewater Marina is located on the western boundary of the city of

Figure 51. Optimum improvement plan at
Chagrin River, Ohio

Cleveland adjacent to Cleveland Harbor. The marina basin, essentially rectangular in shape, accommodated mooring of over 600 boats. Harbor protection was provided by the Cleveland Harbor breakwater on the east and a rubble-mound breakwater (with sheet pile on the marina side) to the north. Occasional rough wave conditions in the marina caused damage to harbor structures and boats moored to the docks, with waves reaching 3 to 4 ft. Proposed improvements at the marina consisted of modifications to the channel entrance with a jetty extension, marina basin modifications which would entail rubble wave absorber along vertical walls in the basin, and/or major structural alteration of the entrance which would involve closing the present entrance and entering through the Cleveland Harbor west breakwater.

203. Model tests were conducted for Edgewater Marina in an existing 1:100-scale model of Cleveland Harbor, Ohio. Tests were conducted to determine the degree of wave protection afforded the basin as a result of the proposed modifications (Bottin and Acuff 1983). Test waves with periods ranging from 6 to 9 sec and heights ranging from 4.7 to 13.4 ft were reproduced from four deepwater directions with swl's of +4.5- and/or +5.6-ft lwd. For an improvement plan to be acceptable, wave heights were not to exceed 1.0 ft for wave conditions occurring during boating season.

204. Model tests were conducted for existing conditions and 24 test plan variations of the three originally proposed marina alternatives. Test results for existing conditions indicated wave heights of 3 ft in the basin during boating season. Significant overtopping of the existing structures and reflections in the entrance and harbor basin were observed. None of the three basic alternatives were effective with regard to meeting the wave height criterion without modifications. All the alternatives required that a portion of the existing Edgewater breakwater (adjacent to an existing sheet-pile wall) be raised or increased in length. Tests indicated that stone absorber would have to be placed adjacent to most of the vertical structures in the harbor entrance for a test plan to be effective. Modifications at the entrance for one of the alternatives are shown in Figure 52.

Vermilion Harbor, Ohio

205. Vermilion Harbor is located on the south shore of Lake Erie at the mouth of the Vermilion River about 37 miles west of Cleveland, OH. The harbor included the lower 3,600 ft of the river, numerous artificial lagoons, and a channel approach from the lake. The project included two parallel piers, with an aggregate length of 2,200 ft, spaced 125 ft apart. The harbor entrance was exposed to storm waves from directions ranging clockwise from west to northeast. Storm waves broke in the relatively shallow water inside and immediately outside the entrance piers, making navigation difficult and dangerous during moderate storms and prevented use of the harbor as a harbor of refuge.

206. A 1:75-scale hydraulic model investigation was conducted to determine wave action in the harbor for existing conditions and develop an optimum breakwater plan (Brasfeild 1970). Waves were reproduced with periods ranging

Figure 52. Rubble-mound modifications required to meet the wave criteria at Edgewater Marina for one of the basic alternative plan configurations

from 5 to 8 sec and heights ranging from 4 to 11 ft for four deepwater directions using a +3.0-ft swl lwd. For an improvement plan to be acceptable, the sponsor specified that wave heights were not to exceed 2.5 ft at the entrance to the harbor or 1.5 ft in the river channel at the entrances to the lagoons.

207. Model tests were conducted for existing conditions and 11 plans of improvement consisting of variations in the number, length, and orientation of cellular steel sheet-pile breakwater structures. For existing conditions, it was confirmed that undesirable wave conditions existed in the entrance channel and the entrances to the lagoons.

208. The originally proposed improvement plan consisted of arrowhead breakwaters, with an aggregate length of 950 ft, that formed a 200-ft navigation entrance. This plan resulted in wave heights up to 5.0 ft in the channel entrance and excessive wave energy several hundred feet up the channel and would not provide sufficient protection for full use of the harbor. It was concluded from test results that an offshore breakwater, approximately 700-ft-long, installed perpendicular to the entrance channel center line and 200 ft from the outer end of the existing east channel pier (Figure 53) was required. It would provide adequate protection from wave action in the harbor entrance, in the lower reaches of the river channel, and at the entrances to the lagoons.

Figure 53. Optimum breakwater plan developed in the model for Vermilion Harbor, Ohio

Ludington Harbor, Michigan

209. Ludington Harbor is located on the eastern shore of Lake Michigan about 60 miles north of Muskegon, MI. The harbor consisted of an outer basin formed by two shore-connected arrowhead breakwaters, an inner harbor which connected the outer basin with the northern end of Pere Marquette Lake, and berthing facilities in Pere Marquette Lake. The existing width and depth of the Ludington Harbor entrance were not adequate to permit safe passage of large vessels into Pere Marquette Lake. Plans were to remove a 100-ft portion of the south breakwater, widen the inner entrance channel, and deepen the channel. Rubble removed from the breakwater would be used to construct a detached breakwater in front of an existing small-boat launching ramp on the north side of the outer harbor.

210. Since widening the breakwater opening and widening and deepening the entrance channel would allow increased wave energy to enter the harbor, a 1:100-scale hydraulic model investigation was conducted to develop optimum design features of the improvement plan with respect to wave and navigation conditions (Crosby and Chatham 1975). Waves with periods ranging from 7 to

9 sec and heights ranging from 6 to 13 ft were reproduced from five directions of wave approach using a +3.8-ft swl lwd. For a plan to be acceptable, wave heights at the inner harbor docks were not to exceed those of the existing harbor.

211. Tests were conducted for existing conditions and seven improvement plans. For existing conditions, most wave heights measured along the docks were 1.5 ft or less. The originally proposed improvement plan resulted in significantly increased wave heights in the entrance channel and in the vicinity of the docks. A 500-ft-long absorber on the south side and a 900-ft-long absorber on the north side of the channel sides (Figure 54) effectively reduced wave heights in the inner entrance channel and in the vicinity of the docks (generally lower than in the existing harbor). The improvements did not significantly change existing current patterns, but magnitudes were slightly increased in the outer harbor. It also was determined that the new breakwater lakeward of the small-boat launching ramp in the outer harbor was effective in reducing wave heights at the ramp to an acceptable level.

New Buffalo Harbor, Michigan

212. New Buffalo, MI, is located at the mouth of the Galien River on the southeast shore of Lake Michigan, about 35 miles east of Chicago, IL. A system of breakwaters had been proposed to protect small-boat mooring facilities constructed upstream of the mouth of the river. The area was exposed to wave action reproduced by storms from directions ranging counterclockwise from north-northeast through west. In addition to a wave action problem, a significant shoaling problem existed due to littoral drift, (the predominant direction was north to south).

213. A 1:75-scale hydraulic model was designed and constructed to determine if proposed improvement plans would provide adequate protection for storm wave conditions (Dai and Wilson 1967). Waves with periods ranging from 7 to 11 sec and heights ranging from 7.0 to 10.0 ft were reproduced from seven directions of wave approach using a +4.5-ft swl lwd. For a plan to be acceptable to the sponsor, wave heights in the outer harbor basin, between the navigation entrance and the mouth of the Galien River, were not to exceed 2.5 ft, and wave heights in the area of a boat-launching ramp inside the inner harbor basin were not to exceed 1.5 ft.

214. Model tests were conducted for 15 test plan configurations. The originally proposed plan included an 861-ft-long rubble-mound south breakwater

Figure 54. Optimum absorber lengths required at inner entrance to Ludington Harbor, Michigan

and a 1,405-ft-long rubble-mound north breakwater which overlapped the south structure, providing a 200-ft-wide entrance. Test results indicated that the originally proposed plan would provide the desired wave protection. A 100-ft-long portion of the outer north breakwater could be removed and also result in wave heights within the established criteria (Figure 55).

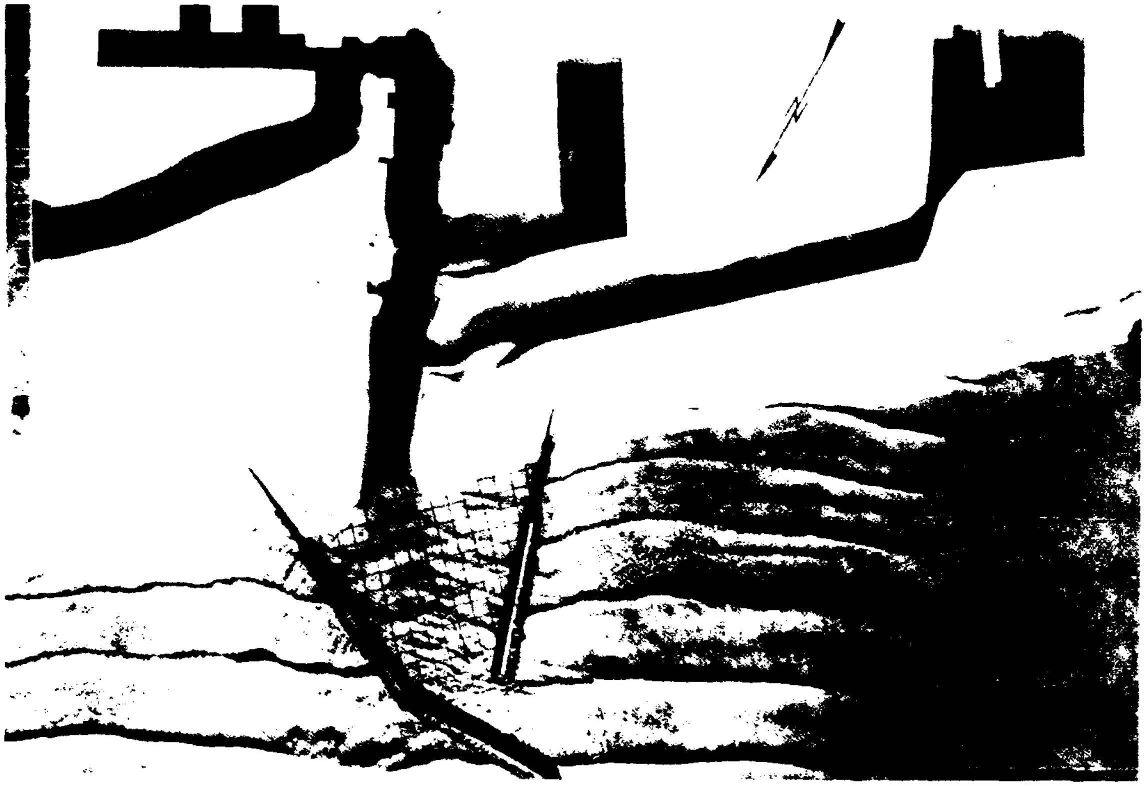

Figure 55. Optimum breakwater alignment for wave protection of New Buffalo Harbor, Michigan

Additional configurations were tested and optimized. It was determined that moving the outer arm of the north breakwater shoreward would provide the desired wave protection with less length, however, navigation conditions would not be as good due to sharper turns the vessels would have to make entering the channel. A plan with a 250-ft-wide channel was optimized at the harbor entrance by using the original north breakwater length and reducing the length, at the lakeward end, of the south breakwater.

Gary Harbor, Indiana

215. Gary Harbor, Indiana, is located at the southern end of Lake Michigan, about 20 miles southeast of Chicago. The harbor consisted of a slip about 5,500 ft long and 250 ft wide, which was partially protected from storm-wave attack by a caisson-type breakwater extending about 3,200 ft from shore in a northeasterly direction. A 9,700-ft-long, vertical-wall, steel sheet-pile bulkhead and landfill were proposed along the lakefront adjacent to the existing slip.

216. A 1:150-scale hydraulic model study was conducted to determine the effect of the proposed bulkhead on wave and current conditions in the area of Gary Harbor and to develop remedial plans for the alleviation of undesirable wave and current effects (Housley 1959). Waves with periods of 7.2 to 9.5 sec and heights ranging from 12 to 13.5 ft were reproduced from four directions using a +3.1-ft swl lwd.

217. Tests were conducted for existing conditions and nine test plan configurations. For existing conditions, wave heights during periods of storm wave attack were almost 4 ft in the harbor slip. Installation of the vertical-wall bulkhead revealed serious reflection problems with respect to navigation. Tests indicated that a 4,450-ft-long rubble-mound absorber would be necessary to prevent an increase in wave action in the navigation areas. Model testing revealed that to significantly reduce wave heights in the slip area, the existing breakwater would have to be extended by 600 ft and an additional breakwater would have to be constructed perpendicular to the outer arm of the existing breakwater (Figure 56). Construction of this configuration would reduce currents transverse to the harbor slip below those measured for existing conditions.

Port Washington Harbor, Wisconsin

218. Port Washington Harbor is located on the west shore of Lake Michigan about 25 miles north of Milwaukee, WI. The harbor was afforded some protection from storm waves by a system of converging rubble and caisson-type breakwaters forming a navigation opening 350 ft wide. The breakwater system was 3,500-ft-long and the harbor covered about 60 acres. Waves, from directions northeast clockwise through south-southeast, had occasionally caused considerable damage to harbor facilities. Waves passed through the navigation opening and traveled along a vertical-wall wharf into the slip areas of the harbor. Also, wave overtopping of the north caisson breakwater generated hazardous conditions in the harbor for moored vessels as well as vessels navigating the entrance.

219. A 1:100-scale hydraulic model investigation was conducted to determine the efficiency of proposed improvement plans and develop new plans, if necessary, to alleviate undesirable wave conditions in the harbor (Fortson et al. 1951). Waves with periods ranging from 5.5 to 7.5 sec and heights from 10.5 to 14.5 ft were reproduced from eight directions of wave approach for a +2.0-ft swl lwd.

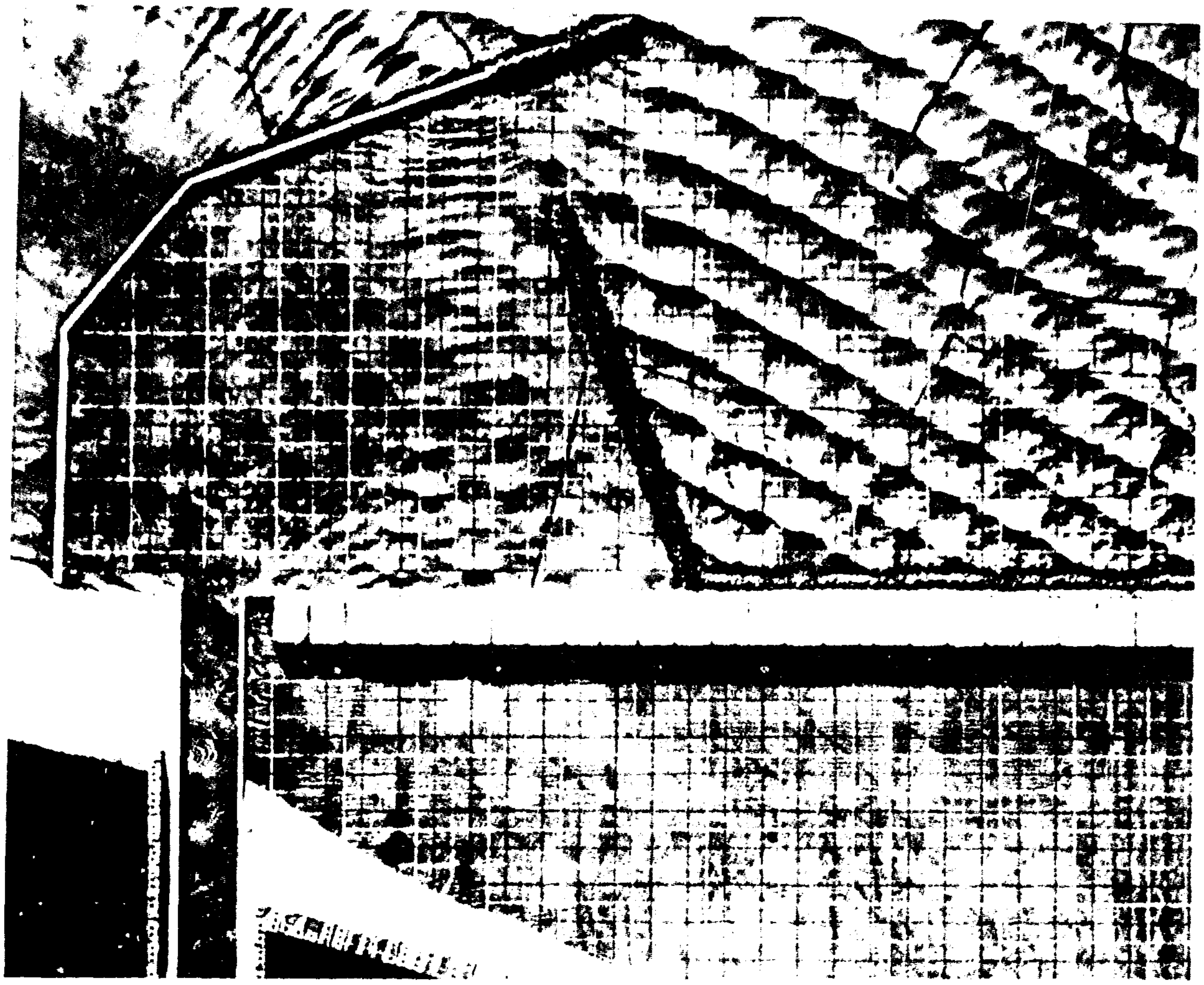

Figure 56. Breakwater modifications required for wave and current protection to Gary Harbor, Indiana

220. Tests were conducted for existing conditions and 29 test plan configurations. The originally proposed configuration consisted of the placement of wave absorber at various locations in the slips and on the lakeside of the north and south caisson breakwaters. The plan was effective in reducing breakwater overtopping, however, energy propagating through the entrance and moving into the inner basins resulted in inadequate protection to the harbor for storm wave action. Numerous plans were tested which included removal of the south breakwater, in conjunction with an offshore structure and a lakeward curved extension of the north breakwater. The optimum plan (Figure 57) entailed a north breakwater extension (1,280 ft) and removal of a portion of the south breakwater. This plan provided adequate wave protection in the inner slip areas.

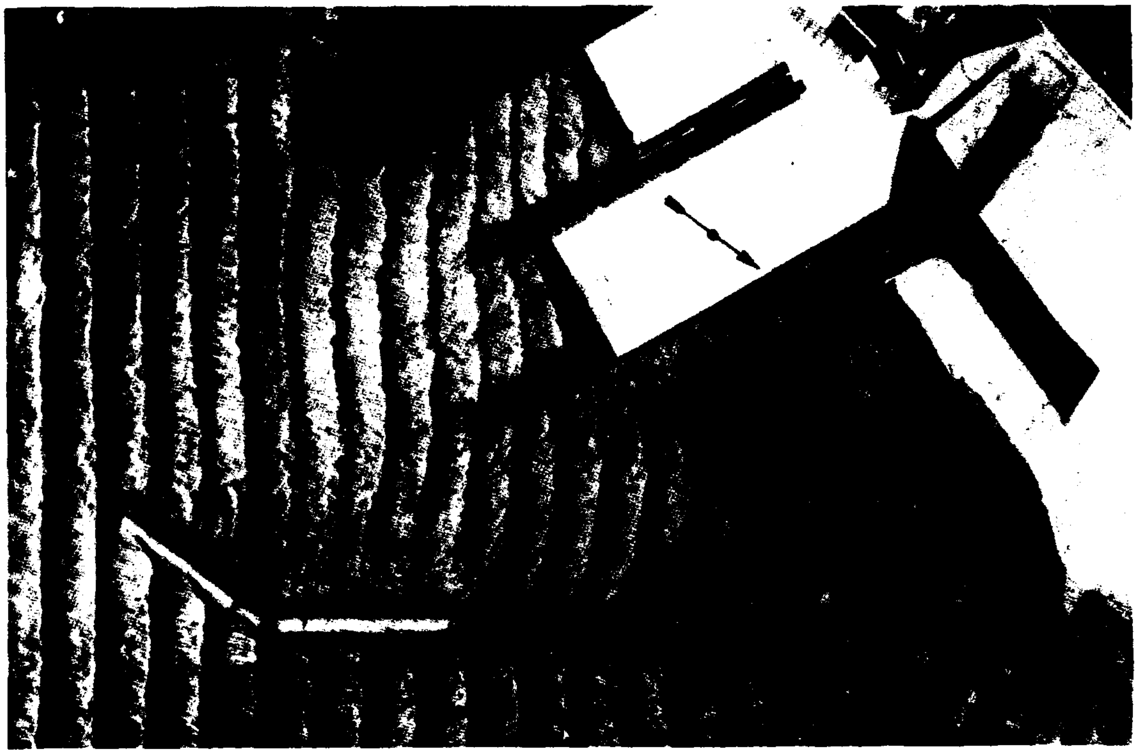

Figure 57. Optimum plan developed in the 1:100-scale model for protection of the inner slips at Port Washington Harbor, Wisconsin

221. Subsequent to the 1:100-scale model study, another model investigation was conducted (at a 1:75 scale) to determine improvements required for a small-boat harbor within the existing outer harbor (Bottin 1977a). Waves with periods ranging from 5.5 to 9.4 sec and heights ranging from 3.4 to 14.7 ft were reproduced from six directions with a +3.9-ft swl. For a plan to be acceptable to the sponsor, wave heights in the proposed small-boat harbor were not to exceed 2.0 ft in the turning basin and 1.0 ft in the mooring areas.

222. The originally proposed improvement plan included a 1,010-ft-long west breakwater, a 320-ft-long east breakwater, and a 500-ft-long wave absorber adjacent to the shoreward side of the existing north breakwater. Test results indicated that the established wave height criterion would be exceeded for test waves from all directions. Observations revealed excessive overtopping of the existing north breakwater (adjacent to the new harbor) and overtopping of and transmission through the proposed east and west breakwaters. It was determined, by testing of numerous improvement plans, that a

+12-ft parapet wall should be installed on the north breakwater adjacent to the small-boat harbor in conjunction with the absorber. Also the new east and west breakwaters must be raised and/or an impervious center added to reduce wave heights to the established criteria. A view of the proposed small-boat harbor within the existing outer harbor at Port Washington is shown in Figure 58. It also was determined that removal of a 185-ft section of the shore end of the west breakwater would improve wave-induced circulation without increasing wave heights in the harbor.

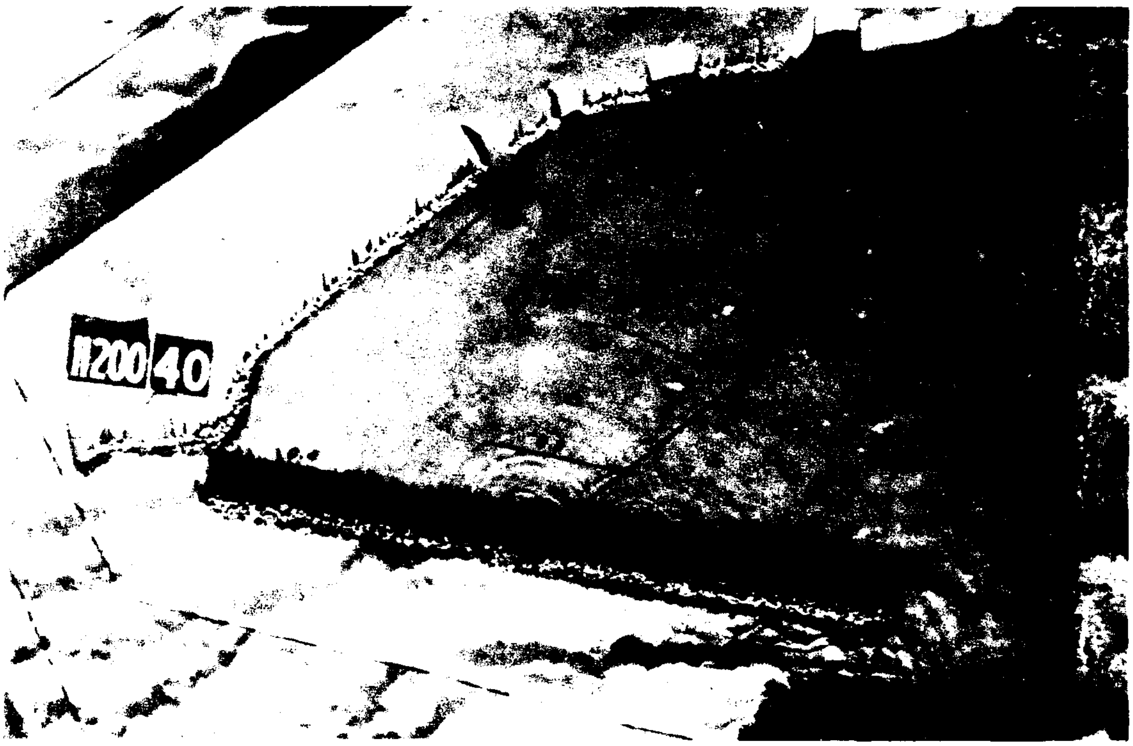

Figure 58. Optimum small-boat harbor configuration developed in the 1:75-scale model of Port Washington Harbor, Wisconsin

223. Additional tests were conducted in the 1:75-scale model of Port Washington to determine the effects of Igloo wave absorber units in the harbor (Bottin 1976). Conclusions drawn from the results of these tests indicated that Igloo wave absorber units placed in and around the slip areas will significantly reduce wave heights in the slips. The east and west breakwaters of the proposed harbor were constructed of Igloos, and it was determined that they would not be stable without a backing structure.

Little Lake Harbor, Michigan

224. Little Lake Harbor is a harbor-of-refuge located on Lake Superior about 30 miles east of Grand Marais, MI. The project consisted of a 1,000-ft-long dogleg west breakwater and a 270-ft-long east breakwater with an 80-ft-wide channel dredged between Little Lake and Lake Superior. Severe shoaling occurred at the harbor entrance that made navigation to the protective harbor both difficult and dangerous even during relatively mild weather conditions. Entrance shoaling was caused by wave action moving material alongshore, and seiche activity in Lake Superior created currents through the entrance and may have influenced sediment movement.

225. A 1:75-scale hydraulic model investigation was conducted to develop the most economical plan of improvement that would minimize channel shoaling without adversely impacting navigation (Seabergh and McCoy 1982). Waves with periods ranging from 5 to 9 sec and heights from 4 to 21 ft were reproduced from six directions of wave approach with a +1.0-ft swl lwd. Equipment also was calibrated and utilized to reproduce seiche activity (seiche heights in Little Lake and seiche currents through the entrance) in the model based on prototype data obtained.

226. Tests were conducted for existing conditions and 15 test plans consisting of structural modifications at the entrance. Existing conditions revealed that shoaling problems existed as a result of the influx of sediment along the east breakwater and eastern portion of the channel. Sediment was deposited by wave action from all directions. For waves from north to west, sediment moved due to the circulation gyre developed in the lee of the west breakwater, and for other wave directions, the incident longshore conditions caused the shoal.

227. The originally proposed improvement plan, consisting of a significant extension (900 ft) of east breakwater, appeared to have created a sediment trap inside the jetties and actually increased shoaling when compared to existing conditions. Plans with structures extending to the bar that bypasses the harbor would intercept sediment movement and shoaling of the entrance occurred. It was determined that a 570 ft structure eastward of the east breakwater would reduce shoaling and provide good bypassing characteristics in both directions. The plan (Figure 59) also reduced seiche oscillations in the harbor and velocities in the entrance channel.

Figure 59. Optimum breakwater configuration for prevention of shoaling at Little Lake Harbor, Michigan

Grand Marais Harbor, Minnesota

228. Grand Marais Harbor is located on the north shore of Lake Superior about 106 miles northeast of Duluth, MN, and was the only harbor of refuge for small craft along 169 miles of rocky coast. Breakwaters had been constructed on both sides of the 480-ft-wide entrance, but these structures did not prevent the propagation of storm waves into the harbor which caused hazardous conditions for small boats. Small craft had to be beached high on the overbank, even during mild storms, to prevent damages.

229. A 1:100-scale hydraulic model study was conducted to study wave action at the site and provide the most suitable means of protec[illegible] the small-craft anchorage basi[illegible] (Schroeder and Easterby 1941). Waves with 6- to 8.5-sec periods and 12- to 18-ft heights were reproduced from three wave directions for a +1.5-ft swl lwd. A view of the model with the existing structures is shown in Figure 60.

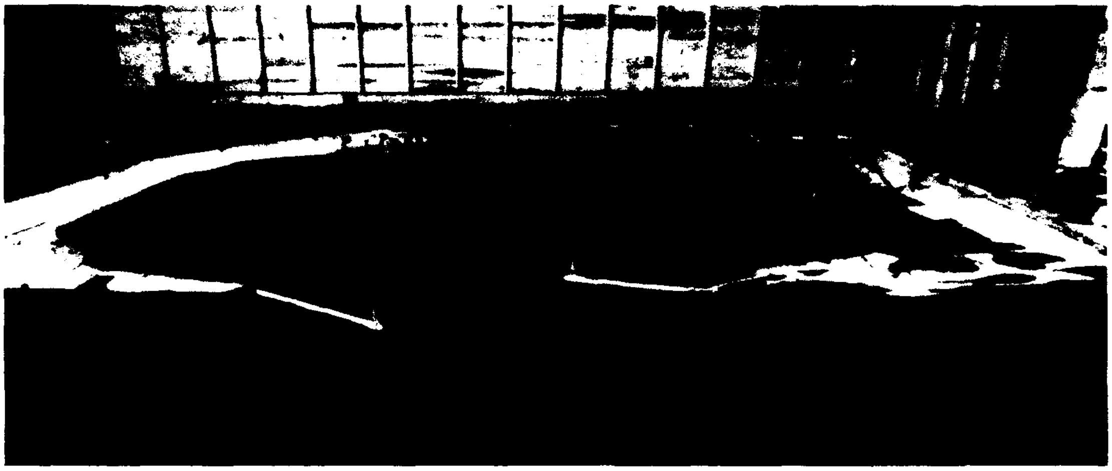

Figure 60. General view of the existing structures in the Grand Marais Harbor, Minnesota, model

230. Tests were conducted for existing conditions and 14 test plan configurations initially. Supplemental tests were conducted for 12 additional plans (Fenwick, Arnold, and Easterby 1944). Tests were conducted for plans which consisted of closing the gaps in the rock ledges located on either side of the harbor entrance; an inner harbor dredged into the shoreline of the main harbor; modifications to the breakwaters at the harbor entrance; and/or an inner harbor formed by enclosing a portion of the main harbor by breakwaters. Test results indicated that none of the modifications of the existing breakwaters or other structures tested at the harbor entrance would be effective in reducing wave action in the harbor. An inner harbor dredged in the west shore of the main harbor and protected at the entrance by short piers resulted in the most favorable wave conditions. The optimum plan, however, considering wave protection and costs, entailed a breakwater in the inner harbor that would provide protection for small craft in its lee, as well as facilities on the north shore.

Discussions

231. Since 1940, 59 hydraulic model investigations of 55 small-boat harbor sites have been conducted at the WES. These studies have been used to optimize structural improvements with regard to hydraulic design and costs. As a result of the model studies, modifications at these harbors have been developed to (a) reduce short-period wave heights in the entrance, navigation

channels, and mooring areas; (b) improve long-period wave conditions in mooring areas; (c) reduce standing waves in harbors; (d) reduce crosscurrents across navigation paths; (e) increase wave-induced circulation inside harbor areas; (f) minimize or eliminate harbor entrance shoaling due to wave-induced longshore currents and tidal currents; (g) stabilize harbor inlet openings; (h) determine locations of deposition basins at inlet entrances; (i) optimize tidal current flows through inlets; (j) improve flood and ice flows through the harbors and their entrances; and (k) determine effects of structures on thermal conditions.

232. Of the 59 model investigations conducted, 17 were conducted for proposed harbor sites where unimproved conditions existed, and 42 were conducted at existing harbor sites where structures existed. Some of these sites included expansions of the existing harbor, however, most studies were conducted to develop remedial plans of improvement where problems existed. Test results for the originally proposed designs for the 59 model studies indicated that 46 proposed designs were ineffective in achieving the desired results, and therefore, subsequent modifications were required in the model to make the design functionally acceptable. Of the 13 originally proposed designs that met the established criteria, 10 projects were over designed. Model tests for these 10 harbor sites indicated the original structure lengths could be reduced, crest elevations lowered, etc. and still provide the desired protection required, thus construction costs were reduced. Of the 59 hydraulic model investigations conducted, only three of the originally proposed designs provided adequate protection without modification. Model tests verified that these improvements could be made without sacrificing harbor protection, and that reductions in structure length, etc., in an effort to reduce costs, could not be made.

233. In review, most of the earlier model studies, generally, were conducted at smaller scales than the more current investigations. It also appears that design conditions of earlier studies were less severe than the design conditions for present studies in similar geographical areas. Some of the earlier studies used impervious breakwaters to represent rubble-mound breakwaters because the transmission of short-period wave energy through these structures was negligible. Subsequent experience has shown that considerable energy may pass through rubble-mound structures, and steps are currently taken, based on scale effect testing conducted at WES, to adjust stone sizes

so that correct transmission and reflection characteristics are reproduced. The earlier studies examined, in most cases, a single problem (i.e. wave conditions). Model studies today are very comprehensive and test numerous conditions (i.e. wave conditions; current conditions; flood, ice, and tidal flows; shoaling conditions; effects of improvements on thermal conditions, etc.). In summary, the capabilities of physical models have been improved through the years, and test conditions have become more representative of prototype conditions through the use of unidirectional and directional spectral wave machines which can produce more realistic wave conditions as compared to old monochromatic wave generating equipment. Also, the use of high-speed computers to control laboratory equipment and collect and analyze data permits studies to consider much larger sets of prototype conditions than could be addressed in model studies prior to the 1970's. Improvements in laboratory sensors have allowed higher resolution and more reliable laboratory measurements and with the advent of laser and ultrasonic sensors detailed measurements of current and velocity fields are now possible where they could only be defined in approximations in the past. The state of the art for hydraulic model investigations is constantly improving.

PART III: PHYSICALLY MODELED HARBORS WHICH HAVE BEEN CONSTRUCTED

Inventory

234. Physical model investigations of 55 small-boat harbor projects in the United States and/or its territories have been conducted at WES. Of the 55 projects, 25 have been constructed in the prototype based on model test results. The physically modeled harbors which have been constructed in the prototype are listed. The number adjacent to the harbor site corresponds to the location shown in Figure 1.

1	Agana Small-Boat Harbor, Territory of Guam
2	Taú Harbor, Island of Taú, American Samoa
3	Waianae Small-Boat Harbor, Oahu, Hawaii
7	Laupahoehoe Point, Hawaii
8	St. Paul Harbor, St. Paul Island, Alaska
9	Siuslaw River, Oregon
12	Crescent City Harbor, California
14	Fisherman's Wharf, San Francisco Bay, California
15	Half Moon Bay Harbor, California
21	Marina del Rey, California
25	Dana Point Harbor, California
30	Murrells Inlet, South Carolina
31	Little River Inlet, South Carolina
32	Masonboro Inlet, North Carolina
39	Port Ontario Harbor, New York
40	Oswego Harbor, New York
43	Cattaraugus Creek Harbor, New York
44	Barcelona Harbor, New York
45	Conneaut Harbor, Ohio
46	Geneva-on-the-Lake Small-Boat Harbor, Ohio
49	Vermilion Harbor, Ohio
50	Ludington Harbor, Michigan
51	New Buffalo Harbor, Michigan
53	Port Washington Harbor, Wisconsin
55	Grand Marais Harbor, Minnesota

Site Specific Construction and Performance

235. In this portion of the report, the 25 small-boat harbor projects which have been modeled at WES and subsequently constructed in the prototype are discussed. A description of the structural features of the prototype harbor are presented and model test plans are reviewed to determine if the prototype harbor modifications were, in fact, those tested and recommended in the model study. Also briefly discussed are the performances of the various harbor sites since construction. A detailed, in-depth, study of harbor performance was not conducted. Information on harbor performance was obtained from various Corps of Engineer District personnel familiar with the projects, the "Monitoring Completed Coastal Projects" (MCCP) program administered by the Corps of Engineers, and local harbor personnel, harbor users, and newspaper articles. The information contained herein gives an indication of how the projects have performed and includes problems that may have been encountered since construction.

Agana Small-Boat Harbor, Territory of Guam

236. In 1977 the existing Agana Small-Boat Harbor was expanded. A 1,135-ft revetted mole was constructed with provisions for additional berthing area on its shoreward side. A sewage treatment plant also was constructed west of the mole. To minimize navigational difficulties, a 525-ft-long west breakwater and a 200-ft-long east breakwater were installed at the entrance. In addition, a 250-ft-long rubble-wave absorber was constructed inside the harbor entrance to dampen wave energy that may otherwise propagate, or be reflected, into the proposed mooring areas. A view of the completed harbor is shown in Figure 61. Prior to the expansion, Agana was experiencing navigational problems due to a sharp reverse bend in the entrance channel and wave-induced crosscurrents.

237. A review of the model investigation of Agana Small-Boat Harbor (Chatham 1975) indicated that a 650-ft-long west breakwater and a 325-ft-long east breakwater were recommended to minimize crosscurrents and confused wave patterns in the entrance. The lengths of each of these structures were 125 ft longer than those built in the prototype. The alignment of the breakwaters constructed at the site, however, were the same as recommended in the model.

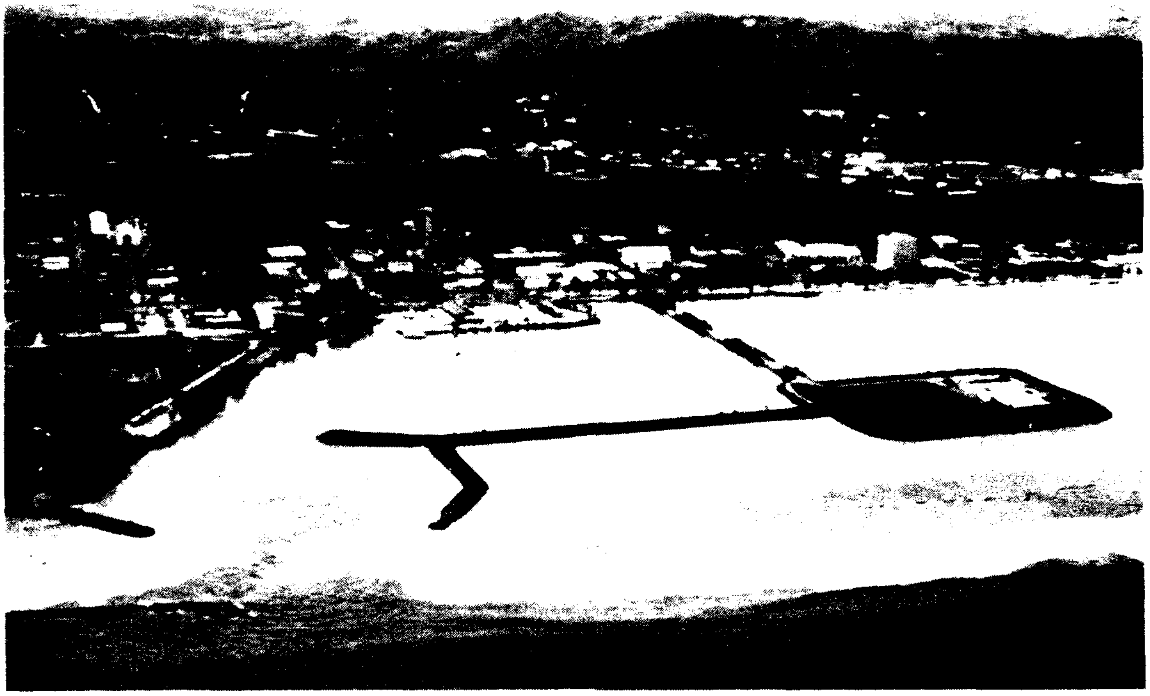

Figure 61. Agana Small-Boat Harbor, Agana, Guam

Also, the model was tested with a 500-ft-long wave absorber inside the entrance as opposed to the 250-ft-long structure constructed in the prototype. An arrangement of open channels was determined in the model to provide adequate circulation in the berthing areas of the harbor. Circulation channels were constructed in the prototype; however, the proposed new mooring areas tested in the model have not yet been developed at the site. The sewage treatment plant constructed at the site (west of the mole) was the same as tested at a seaward location in the model. A shoreward location was also tested.

238. An assessment of post construction performance of wave conditions in the proposed mooring areas cannot be made since this development has not yet been constructed. No berthing problems have been encountered in the existing small boat mooring area; however, this area is well protected from waves by the new breakwaters and mole section. Even though the breakwater lengths constructed in the prototype were shorter than those tested in the model, the breakwaters are very effective in minimizing crosscurrents and improving navigation conditions in the new entrance. The breakwaters tend to

deflect wave-induced currents moving along the reef in a seaward direction, thus preventing crosscurrents in the channel. The small craft using the existing berthing areas at Agana are not experiencing navigation problems at the entrance. Current conditions in the harbor entrance appear to be reacting as predicted by the model. It has also been noted that the channels constructed at the site are very effective in providing circulation in the proposed berthing areas.

Taú Harbor, Island
of Taú, American Samoa

239. Construction of Taú Harbor was completed in 1981. It consisted of a rectangular basin enclosed by a revetted landfill and a 290-ft-long breakwater on the east and a 200-ft-long groin on the west (Figure 62). The harbor was constructed for the loading and unloading of cargo and personnel transported between the islands by an interisland tug and barge network. No harbor existed on the island until construction of Taú Harbor.

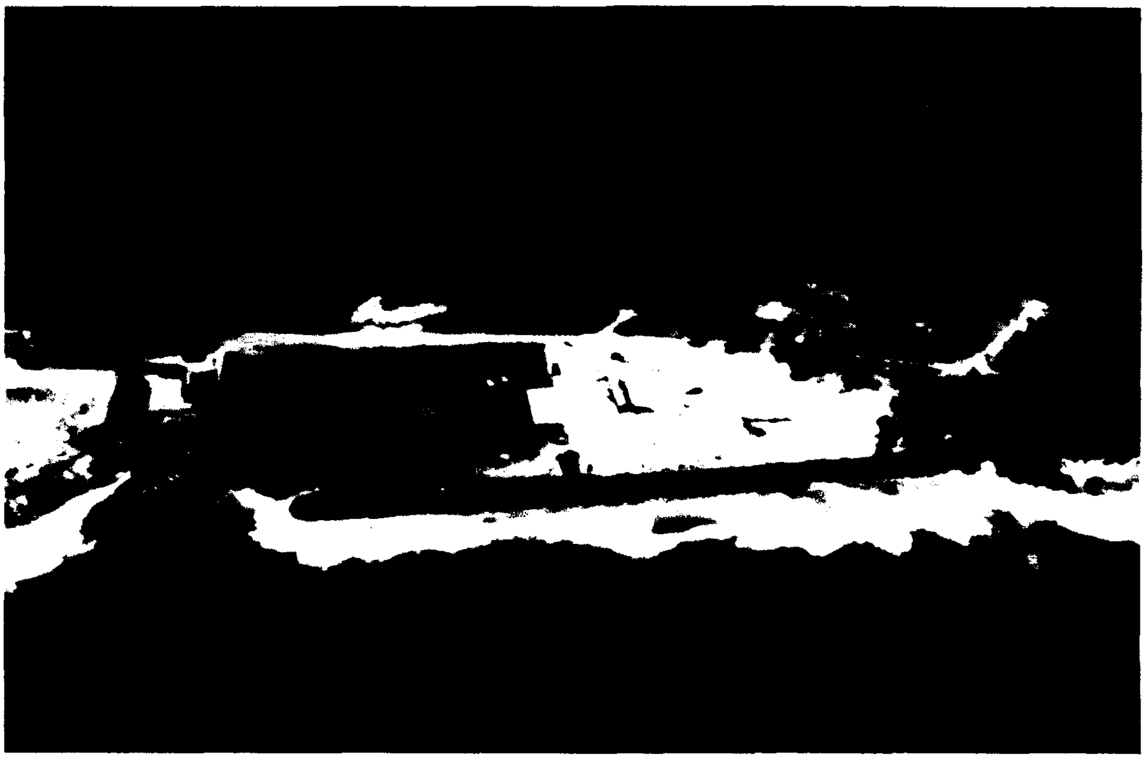

Figure 62. Tau Small-Boat Harbor, Taú Island, American Samoa

240. A review of the model investigation of Taú Harbor (Crosby 1974) revealed that the harbor structures were constructed in the prototype based on

the plan recommended by the model study. It is noted, however, that the entrance channel tested in the model was 100 ft wide, 14 ft deep, with a 12-ft-deep turning basin. In the prototype, the entrance channel is 130 ft wide and 16 ft deep and the turning basin was dredged to a 14-ft depth. None of the plans tested in the model satisfied the 3.0-ft wave height criterion in the berthing area for all test waves; however, the plan that was constructed in the prototype came closest to meeting the criterion. The harbor was not designed to be an all-weather harbor and conclusions of the study stated that the plan would be acceptable only if it was recognized that there would be periods when the harbor would not be usable. The model also indicated wave-induced currents in the entrance ranging up to 4.7 fps for 5-ft incident waves and up to 7.1 fps for 15-ft incident waves. It was also recognized in the study that when waves are breaking across the entrance or crosscurrents are present, vessels would not be able to leave or enter the harbor.

241. A review of the performance of Taú Harbor since construction reveals that vessels experience navigation problems in the entrance channel. Waves approach and break over the reef on both sides of the entrance channel and spill into it making it difficult for vessel operators to identify the channel limits. Wave-induced currents, due to waves breaking across the reef, also present navigational difficulties. The use of the harbor is limited due to these phenomena. Excessive wave heights in the berthing area also occur at times, but are not as prohibitive as the undesirable conditions in the entrance. The model study indicated that the plan built in the prototype was the best that was tested, but it also predicted excessive wave heights in the berthing area at times and periods when navigation conditions would not be favorable due to breaking waves and crosscurrents in the entrance channel. For available funds, the best harbor configuration was constructed, but it is not, nor was it recognized as, an all-weather harbor.

Waianae Small-Boat Harbor, Oahu, Hawaii

242. Construction of a 1,690-ft-long main breakwater and a 220-ft-long stub breakwater at Waianae were completed in 1979 (Figure 63). The Waianae coast provides some of the best fishing in the Hawaiian islands and is an excellent boating area. The new harbor provided much needed small-craft berthing facilities in this vicinity.

243. A review of the model investigation at Waianae (Bottin, Chatham and Carver 1976) indicated that the prototype harbor was constructed similar

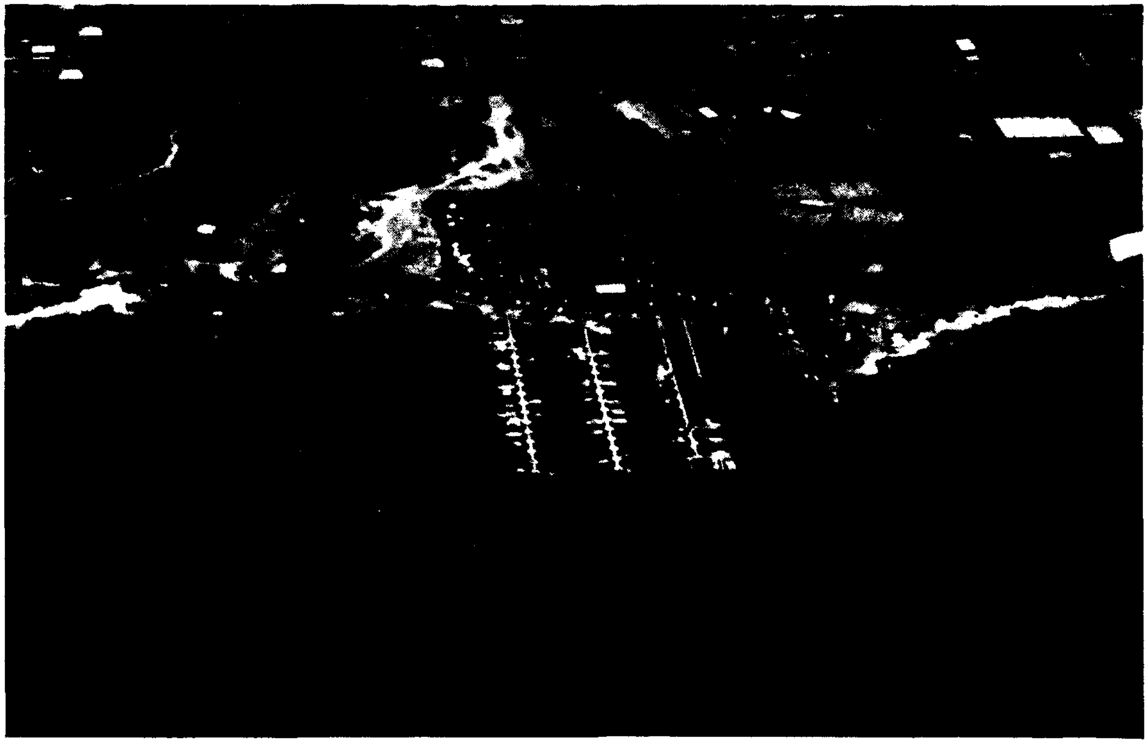

Figure 63. Waianae Small-Boat Harbor, Oahu, Hawaii

to the optimum configuration tested in model, but discrepancies did exist. The model revealed an optimum outer breakwater length of 1,650 ft and a spur breakwater length of 250 ft. The outer breakwater constructed at the site was 40 ft longer than recommended in the model, and the stub breakwater was 30 ft shorter than that recommended. The longer outer breakwater provided more overlap of the entrance, but the shorter stub structure resulted in an entrance opening that was wider than recommended. The breakwater cross sections and crest elevations recommended in the model were the same as those constructed in the prototype. The plan was developed in the model to achieve a wave height criterion of 2.0 ft in the berthing areas.

244. A review of the performance of Waianae Harbor after construction reveals complaints relative to the harbor. There are no indications of vessel or facility damage, but excessive movement of the vessels in the slip areas is a nuisance and aggravating to boat owners. Complaints have also been received regarding undesirable wave conditions at the boat ramp located immediately inside the entrance. The harbor users' complaints are not during periods of storm conditions but for conditions that occur on a day-to-day basis.

Excessive wave energy enters the harbor for normal seas. The wider entrance channel width constructed in the prototype may contribute to some of the excess wave energy entering the harbor, however, the selected 2.0-ft wave height criterion that the harbor was designed for was probably excessive. Currently, most small-boat harbors are currently designed for a 1.0-ft wave height criterion in the mooring areas during storm wave conditions. A review of the model tests indicates that for 4-ft incident test waves (common conditions), wave heights in the mooring areas could range from 1.1 to 1.6 ft depending on the wave period and direction of approach. For storm waves (8 to 12 ft in height), the model met the criterion of 2.0 ft, but for more common conditions wave heights up to 1.6 ft could occur (greater than the current criteria for storm conditions in most small-craft harbors). Most Hawaiian small-boat harbors also have fixed docks, since the tidal range is relatively small. With the mooring line configuration at Waianae, vessels are taunt at low tide, but at high water their lines have slack which allows more freedom for movement. Small craft in the harbor, therefore, move more in their slips at the higher water levels as they ride the crests of the incoming waves. The model indicated wave heights ranging from 1.0 to 1.8 ft at the boat launching ramp for 4-ft incident wave conditions from the most predominant wave direction. Prototype conditions may result in greater heights, however, since the stub breakwater at the site is shorter than that recommended in the study, which may allow more wave energy to the ramp location. Excessive wave heights at Waianae do occur for normal wave conditions, but upon close examination, the model tests predicted these. The primary problem was that the 2.0-ft design wave height criterion probably was too high, especially with the slack mooring line conditions that occur during the tidal cycle due to the fixed, as opposed to free floating docks. The boat launching ramp also should be in a more protected area in the harbor than its current location directly inside the entrance.

Laupahoehoe Point, Hawaii

245. In 1988, improvements were completed at Laupahoehoe Point, Hawaii, to provide a protected boat-launching ramp. Construction of a 200-ft-long rubble-mound breakwater (seaward end having a rib cap and armored with dolosse), a 60-ft-long wave absorber installed adjacent to the shoreline, an entrance channel, and a turning basin were included. These improvements would allow local fishermen to take full advantage of the ocean's resources in the

immediate area. A view of the completed project is shown in Figure 64. Prior to improvements, the project was model tested.

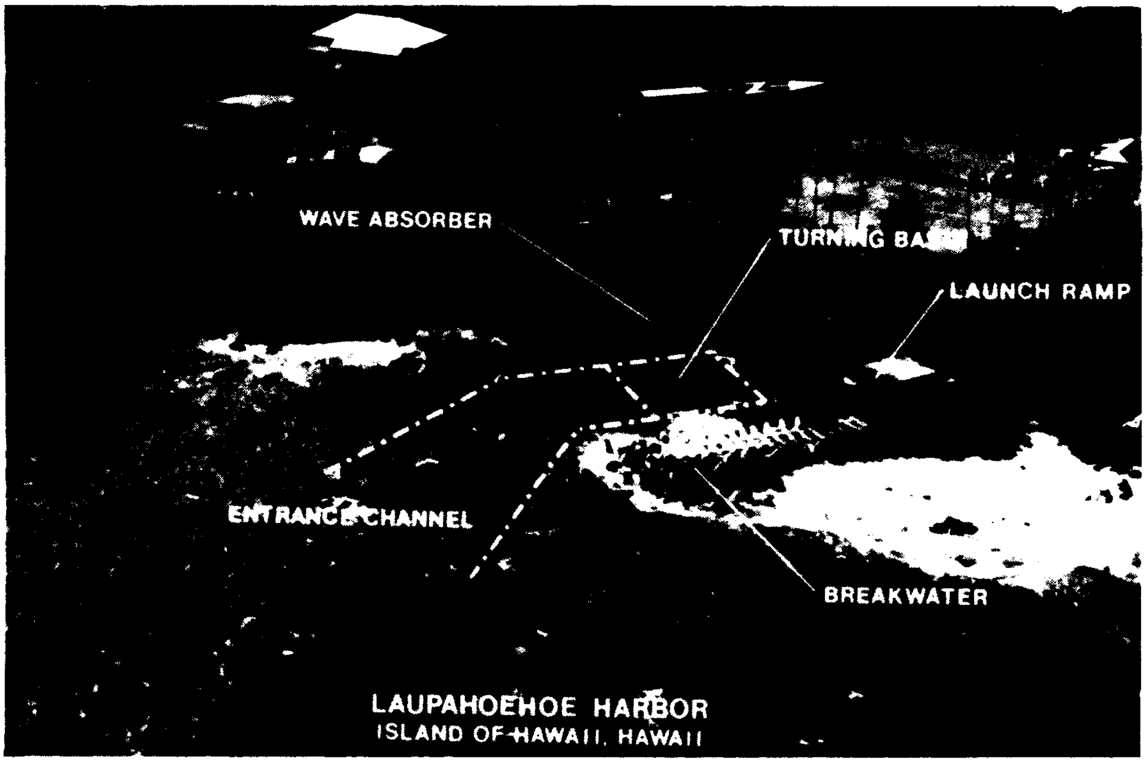

Figure 64. Laupahoehoe Boat Launching Facility, Hawaii

246. A review of the model investigation at Laupahoehoe Point (Bottin, Markle, and Mize 1987) indicated that the proposed project was constructed as recommended in the model. Wave heights at the boat ramp were not to exceed 2 ft for normal wave conditions (deepwater waves of 6 ft or less). Wave heights within this criterion were expected to render the launching ramp usable approximately 70 percent of the time. Due to year-round rough water conditions, a facility designed for 100-percent usage was not economically feasible.

247. The performance of the Laupahoehoe Boat Launching Facility since construction indicates that the ramp is not usable a large portion of the time. Laupahoehoe is in an area of relatively high wave energy. Waves with heights of 4 to 6 ft are normal and occur almost constantly at the site. The launching facility was designed with a wave height criterion of 2.0 ft for normal wave conditions. Observations in the prototype indicate that waves ranging from 1.5 to 2.0 ft result in breaking conditions on the boat ramp that

cause unsafe launching conditions. The model predicted wave heights of 2.0 ft or less for approximately 70 percent of the time (based on hindcast data). Since the boat ramp can rarely be used when 2-ft waves are present, due to breaking conditions, it appears that the established wave height criterion was excessive. Despite the fact that the boat launching ramp is not usable as great a percentage of time as expected, local users appear to be content with the project for the benefits that are derived (mostly fishing benefits).

St. Paul Harbor, St. Paul Island, Alaska

248. A breakwater was constructed at St. Paul Harbor during the early 1980's but subsequently failed during storms of 1984. A new breakwater was completed at the site in 1985. This breakwater was 750 ft in length. It functioned well with regard to stability, but was not of sufficient length to provide wave protection to vessels utilizing a dock, which was adjacent to the harbor side of the structure. In 1989, construction of a breakwater extension and a secondary breakwater at St. Paul Harbor was completed. The breakwater extension was 1,050 ft in length and the detached secondary breakwater was 1,000-ft-long. A view of these structures while under construction is shown in Figure 65. The 1989 improvements were model tested prior to construction.

Figure 65. St. Paul Harbor structures during construction

249. A review of the model investigation report (Bottin and Mize 1988) revealed that the project was constructed as recommended by the model with the exception of the length of the secondary breakwater. The secondary structure recommended in the model was 1,100 ft long as opposed to the 1,000-ft-long breakwater that was constructed. The prototype breakwater was 100 ft shorter than the recommended one at its seaward end resulting in a navigation opening of 350 ft, as opposed to the 250-ft-wide opening developed in the model. The outer breakwater extension was constructed as recommended in the model, as well as, the cross sections of both structures.

250. An evaluation of wave and shoaling conditions at St. Paul Harbor since construction indicates that the harbor is performing very well. The site has experienced three to four storms in the winters of both 1989 and 1990. During storm conditions, wave activity along the dock in the lee of the breakwater extension was calm. Local harbor personnel indicated that wave conditions were less than expected, and vessels did not have to move to the designated area in the lee of the secondary breakwater (an area behind the secondary breakwater has been designated for mooring during severe storm conditions). It appears, at this point, that the prototype harbor is performing as predicted in the model study, however, it has probably not yet been subjected to design storm conditions. With the wider entrance constructed in the prototype, wave conditions may be somewhat higher than predicted. The gap between the shoreward end of the detached breakwater and the shoreline also appears to be providing harbor circulation as indicated by the model. In addition, as predicted, no shoaling has occurred in the mooring areas at this point.

Siuslaw River, Oregon

251. Construction of north and south jetties at the mouth of the Siuslaw River was initiated in 1893, and periodic extensions have occurred through 1985. In 1985, the latest extensions of the north and south jetties were completed. They were extended by 1,900 and 2,300 ft to lengths of 9,990 and 6,245 ft, respectively. Also constructed were two 400-ft-long spurs, originating 900 ft shoreward of the head of each jetty, at 45-deg angles to the axis of the jetties on their seaward sides (Figure 66). The purpose of the spurs was to deflect sediment and minimize shoaling of the navigation channel.

Figure 66. Siuslaw River Entrance, Oregon

252. Prior to the most recent jetty extension and spur construction, a model investigation was conducted (Bottin 1981). A review of the model tests results for Siuslaw River indicated that the jetty extensions and spur orientations tested in the model were the same as those constructed in the prototype. Model tests indicated, qualitatively, that material in the nearshore zone would move toward the jetties and into an eddy, created by the spurs, which tended to deflect material away from the structures. Under certain conditions, some sediment would be carried around the end of the spurs and into the V-shaped area formed between the spurs and the jetty trunks, and some material would continue to migrate around the jetty head and into the entrance.

253. After the 1985 construction, the project was selected for the MCCP program administered by the Corps of Engineers. Monitoring of the project is still in progress, however, data to date indicate that the north spur jetty deflects longshore currents from the north into a clockwise eddy north of the structures. Bathymetry data also indicates that underwater contours are orienting themselves parallel to the north jetty and that sediment is

accumulating around the head of the spur with some being driven, by wave energy, into the "V" formed by the spur and the jetty trunk. Quantities of material moving around the head of the north jetty into the entrance channel are not certain. Dredging of the channel has occurred at regular intervals, but volumes of material dredged have not yet been studied. In addition, sediment sources in the entrance contributed by river flows have not been determined. In summary, it appears that current and sediment patterns north of the north jetty and spur are reacting similar to those predicted in the model. At this point, it is inconclusive whether sediment movement into the entrance channel has been minimized as predicted in the model study since the volumes and sources of sediment in the entrance have not yet been studied. Sediment movement, based on bathymetric data, along the south jetty is similar, however, it is more dominant on the north since predominant wave energy is from a northwesterly direction.

Crescent City Harbor, California

254. Between 1920 and 1957 various stages of construction of the Crescent City Harbor were completed which consisted of a 4,670-ft-long outer breakwater, a 1,200-ft-long inner breakwater, and a 2,400-ft-long rubble-mound barrier to prevent sand movement into the inner harbor. With this configuration, however, the harbor was exposed to large waves that caused damage to moored vessels. In 1974 a 400-ft-long extension of the inner breakwater was completed to provide additional wave protection to the mooring area (Figure 67). The inner breakwater extension was modeled prior to construction.

255. A review of the model investigation of Crescent City Harbor (Senter and Brasfeild 1968) reveals that the 400-ft-long inner breakwater extension constructed in the prototype was recommended by the study. The extension should reduce wave heights in the inner harbor to 2 ft or less during storm wave activity with the exception of about 10 hr/yr. With this modification, however, the model indicated that undesirable navigation and wave conditions may still exist in other portions of the harbor. Additional recommendations were made to improve overall conditions in the harbor, but have not been constructed.

256. Over the years stability problems have occurred with the Crescent City Harbor outer breakwaters and monitoring of the project with instrumented dolos armor units was carried out during the period 1986 to 1988 and low level monitoring of insrumented dolos will continue through September 1992. As far

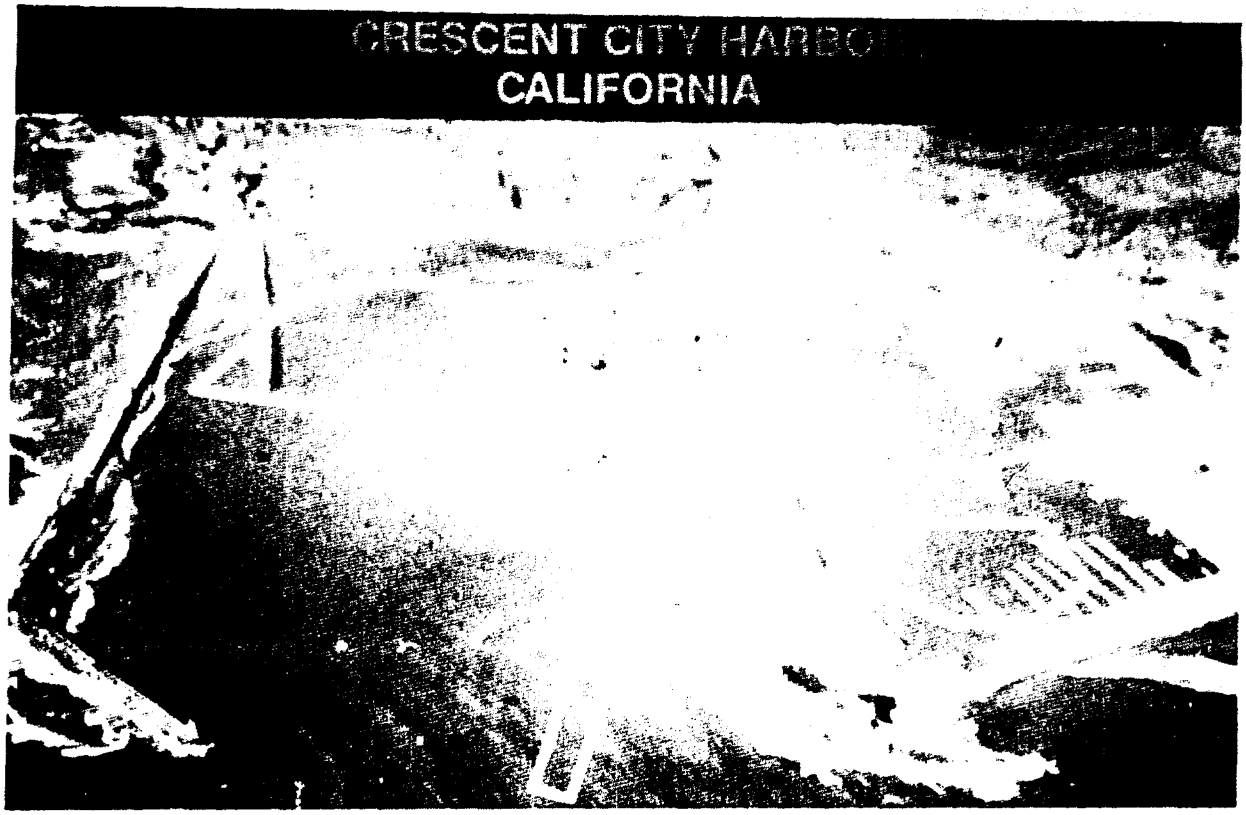

Figure 67. Crescent City Harbor, California

as problems associated with adverse waves and/or vessel damage in the mooring area, however, none have been reported since construction of the 400-ft-long inner breakwater extension. The mooring area appears to be quiet during storm wave activity as predicted by the model study.

Fisherman's Wharf,
San Francisco Bay, California

257. Construction of a 1,509-ft-long solid, concrete-pile breakwater was completed at Fisherman's Wharf in 1986. In addition, two segmented breakwaters (28-ft solid walls with 6-ft openings) were constructed. These latter two structures were 150 and 258 ft long and were built along Pier 45. The breakwater system (Figure 68) was constructed to provide wave protection to the harbor during storm wave conditions. The segmented structures permitted tidal currents to pass through the harbor for flushing. Prior to construction, wave energy resulted in continual damage to fishing vessels and mooring facilities in the area. The harbor configuration was model tested to optimize structure designs prior to construction.

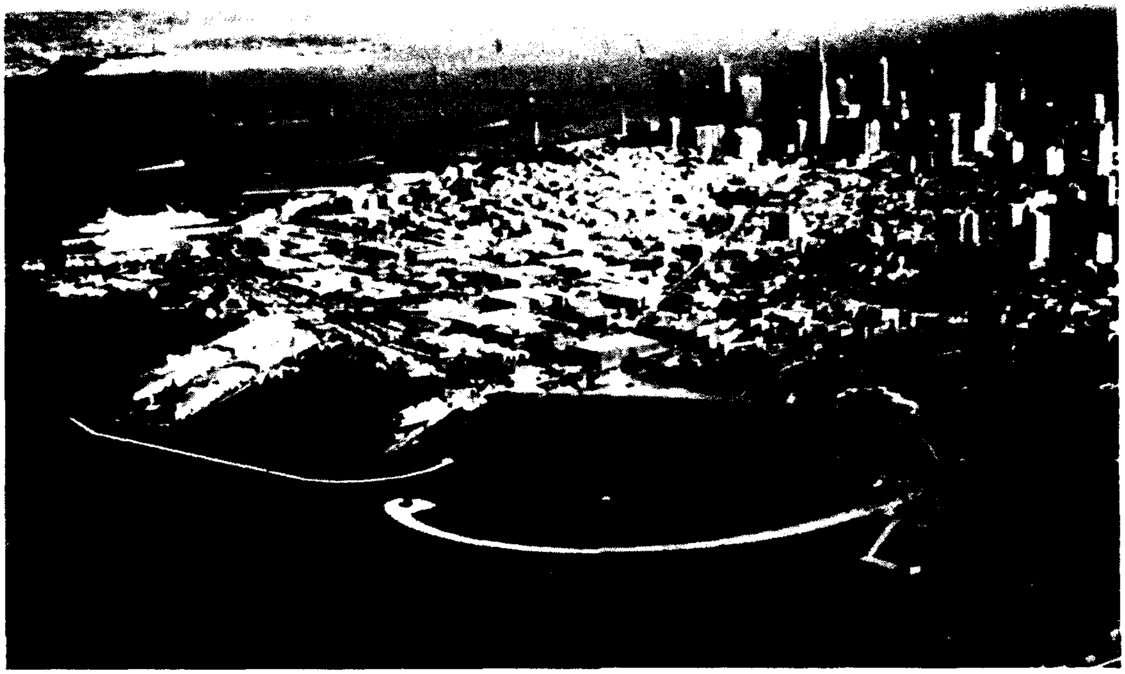

Figure 68. Fisherman's Wharf Harbor, San Francisco, California

258. A review of the model investigation of Fisherman's Wharf Area (Bottin, Sargent, and Mize 1985) revealed that the prototype harbor was constructed similar to the configuration developed and recommended in the model. A 1,585-ft-long solid breakwater was recommended in the model study, as opposed to the 1,509-ft-long structure constructed in the prototype. The end points of the prototype harbor breakwater, however, were in the exact relative locations as those tested in the model. The length difference resulted because the curved breakwater tested in the model was replaced with short straight sections in the prototype. The location of the segmented structures constructed in the prototype were the same as in the model study. Since the end points of the solid breakwater were in the same relative locations and the segmented structures were the same, the results from the model study should be applicable when compared to the prototype.

259. Since construction of the project in the prototype, several storms have occurred and wave conditions along the historic fleet mooring area and the inner basins have been relatively calm with no reports of damage. The project won an engineering design Award of Merit in the 1989 Chief of

Engineers Design and Environmental Awards Program. It was stated that the modeling effort yielded a highly efficient system of three separate breakwaters which minimize physical size and cost, yet provide protection from storms from several different directions. The project was also selected for the MCCP program. Monitoring of the project is currently in progress, and data obtained to date indicate that the completed project is performing as intended.

Half Moon Bay Harbor, California

260. In 1961, construction of two rubble-mound, shore-connected breakwaters was completed at Half Moon Bay Harbor to protect the harbor during severe storms. The west breakwater was 2,620 ft long, and the east breakwater was 4,420 ft long. After construction, however, storm waves frequently caused the berthing areas to be unusable and indicated the structures did not provide adequate wave protection. A 1,050-ft-long extension of the west breakwater was completed in 1967 to alleviate undesirable wave conditions in the harbor (Figure 69). The breakwater extension was modeled prior to construction.

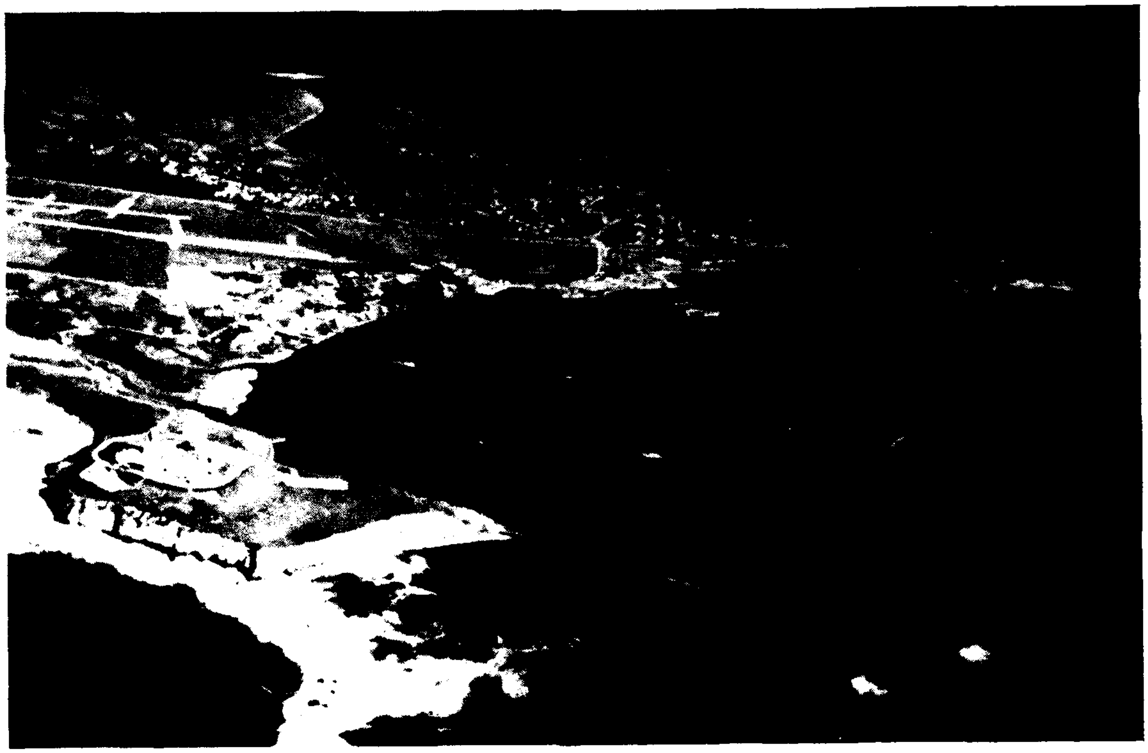

Figure 69. Half Moon Bay Harbor, California

261. A review of the model investigation of Half Moon Bay Harbor (Wilson 1965) indicated that the 1,050-ft-long structure was recommended to

provide the desired protection in the prototype. The length and orientation of the breakwater extension was constructed in the prototype as recommended by the model. The extension was predicted to reduce wave heights to within a 2.0-ft wave height criterion established by the sponsor for storm wave conditions.

262. After construction of the 1,050-ft-long breakwater extension at Half Moon Bay Harbor, small boats were provided adequate protection for open mooring in the harbor. The 2-ft wave height criterion in the harbor, however, was not adequate for mooring small craft in a marina type environment. Subsequent to construction of the breakwater extension, the San Mateo Harbor District developed a marina inside the harbor which was enclosed by rubble-mound breakwaters. The marina was designed so that maximum wave heights would not exceed 1.0 ft during storm wave conditions. The model prediction of 2.0-ft waves in the harbor were correct according to prototype observations during storm events.

Marina del Rey, California

263. In the early 1960's two parallel jetties about 2,000 ft long were constructed to provide protection to Marina del Rey, a small-craft harbor developed by dredging a 2-mile-long channel and eight lateral basins off the main channel. After construction of the marina, waves entering the entrance resulted in intolerable wave conditions in several of the basins. Subsequently, a 2,330-ft-long detached breakwater was constructed (Figure 70) to provide the desired wave protection. The detached breakwater was modeled prior to construction.

264. A review of the model investigation of Marina del Rey (Brasfeild 1965a) revealed that the detached structure built in the prototype was essentially the same as that recommended in the model. The only difference is that the length of the prototype structure was 5 ft longer than that recommended (2,330 versus 2,325 ft). The location, orientation, and wing lengths, etc. were constructed as recommended. (The additional 5 ft of structure was added to the southern wing.) The model study did, however, recommend also that the middle jetty be sealed to an elevation of +8-ft mllw. It was predicted that these improvements would reduce waves in the harbor areas to heights of less than 2 ft.

265. A review of the performance of Marina del Rey reveals that the detached structure has been effective in reducing wave conditions in the

Figure 70. Marina del Rey, California

harbor mooring areas to tolerable levels. Marina del Rey is considered the world's largest man-made small-craft harbor with approximately 6,000 berthing slips. Since construction of the detached breakwater, with the exception of some congestion in the entrance, problems have not been encountered in the harbor. The harbor has performed as predicted by the physical model study.

Dana Point Harbor, California

266. Construction of Dana Point Harbor was completed in 1968. It consisted of a 5,500-ft-long west breakwater, a 2,250-ft-long east breakwater, and inner harbor berthing areas partially enclosed by the shoreline and mole sections (Figure 71). The harbor was constructed in a sheltered cove in the lee of Dana Point and provides mooring facilities for about 2,150 small boats.

267. Prior to construction of the harbor, a model investigation was conducted (Wilson 1966). A review of the study reveals that the harbor was built in accordance with the recommendations of the model. The model predicted that wave heights in the berthing areas would not exceed 1.5 ft during storm wave activity. It was determined in the model tests, however, that at infrequent intervals, and for short durations, wave heights in the fairway

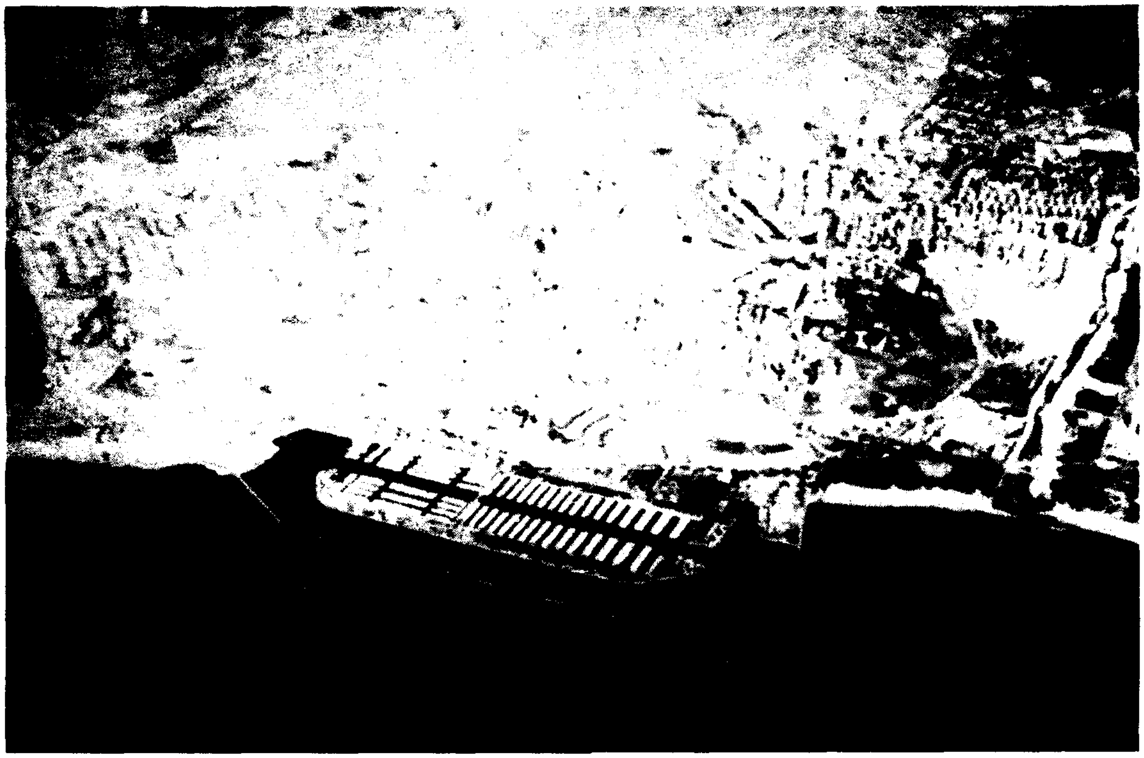

Figure 71. Dana Point Harbor, California

between the west breakwater and the mole could be severe (i.e. about 8 ft) due to wave overtopping of the breakwater.

268. An assessment of the post construction performance of wave conditions at Dana Point Harbor reveals no wave related problems in the mooring areas. Construction of the mooring areas shoreward of the moles appears to be a good harbor design. The outer west breakwater is overtopped by storm waves, which propagate into the revetted moles. Vessels in the mooring areas in the lee of the moles remain protected. During periods of storm wave activity, when other small-boat harbor facilities and small craft in Southern California have received severe damages, Dana Point has reported none. The project is performing as predicted by the model investigation.

Murrells Inlet, South Carolina

269. During the period 1977 to 1981, design improvements were made at Murrells Inlet. They consisted of a north jetty with a low-crested weir section, a south jetty, and sand dikes composed of dredged material which tied the structures into the existing dune lines (Figure 72). Prior to improvements, difficult and dangerous navigation conditions existed due to shifting

Figure 72. Murrells Inlet, South Carolina

shoals and breaking waves.

270. Before improvements were constructed, a model investigation was conducted (Perry, Seabergh, and Lane 1978). A review of the study reveals some discrepancies between the recommended improvements and the structures actually constructed in the prototype. The model recommended a 3,455-ft-long north jetty with a 1,330-ft-long low-crested weir section. In the prototype, a 3,420-ft-long north jetty was constructed with a 1,350-ft-long low-crested weir section. The elevations of both the north jetty (+9-ft mean low water (mlw)) and the weir section (+2.2-ft mlw) constructed were the same as recommended in the prototype. The length of the south jetty construction wa' 3,330 ft versus 3,320 ft recommended in the model, both with elevations of +9-ft mlw. The sand dikes connecting the shoreward ends of the north and south jetties to the existing dune line were recommended, as well as the 600 ft spacing between the jetties constructed in the prototype. The model results also recommended a variable height training dike (el +2.3- to

+9-ft mlw) approximately 1,500 ft long to prevent the migration of tidal flow through the deposition basin. The training dike was not constructed in the prototype. In summary, most of the improvements constructed were similar to those recommended in the model. The jetty spacing and elevations were the same, only the lengths are slightly different. The major discrepancy was the absence of the training dike that was recommended in the model.

271. A review of the prototype performance of the Murrells Inlet's structures reveals that the channel between the jetties has remained stable. The channel maintains its authorized depth and no channel dredging in this area has been required since construction. As intended, sediment from the north shore moves over the weir section in the jetty with the majority settling into the deposition basin, which is dredged periodically. A problem, however, has been encountered shoreward of the deposition basin. At higher tide levels, some material passing over the weir section of the north jetty migrates shoreward due to wave penetration. The sediment is influenced by tidal currents in the inlet channel extending northward. A meandering unstable channel condition occurs in this location. At times a tip shoal forms that directs ebb tidal flows toward the south overbank where scouring occurs. When this condition exists, dredging of the shoal is required. Had the training dike, recommended in the model study, been installed in the prototype, it would have prevented the influx of sediment into the problem area. It was predicted in the study that the training dike also would result in a more stable channel alignment in the area.

Little River Inlet, South Carolina

272. During the 1981-1983 time frame, navigation improvements were constructed at Little River Inlet (Figure 73). These improvements consisted of a 3,300-ft-long east jetty and a 3,800-ft-long west jetty. The jetties consisted of rubble-mound portions with weirs and sand dikes which connected the shoreward ends of the jetties to the shoreline. The seaward ends of the jetties were constructed parallel to each other and situated 1,000 ft apart. A 300-ft-wide navigation channel also was included between the jetties, and a sandfill area was constructed adjacent to the sand dike portion of the west jetty with dredged material. Before construction of the improvements, the inlet was unstable with a constantly changing configuration. A narrow navigation channel and shallow bar regions resulted in difficult and dangerous navigation into the embayment.

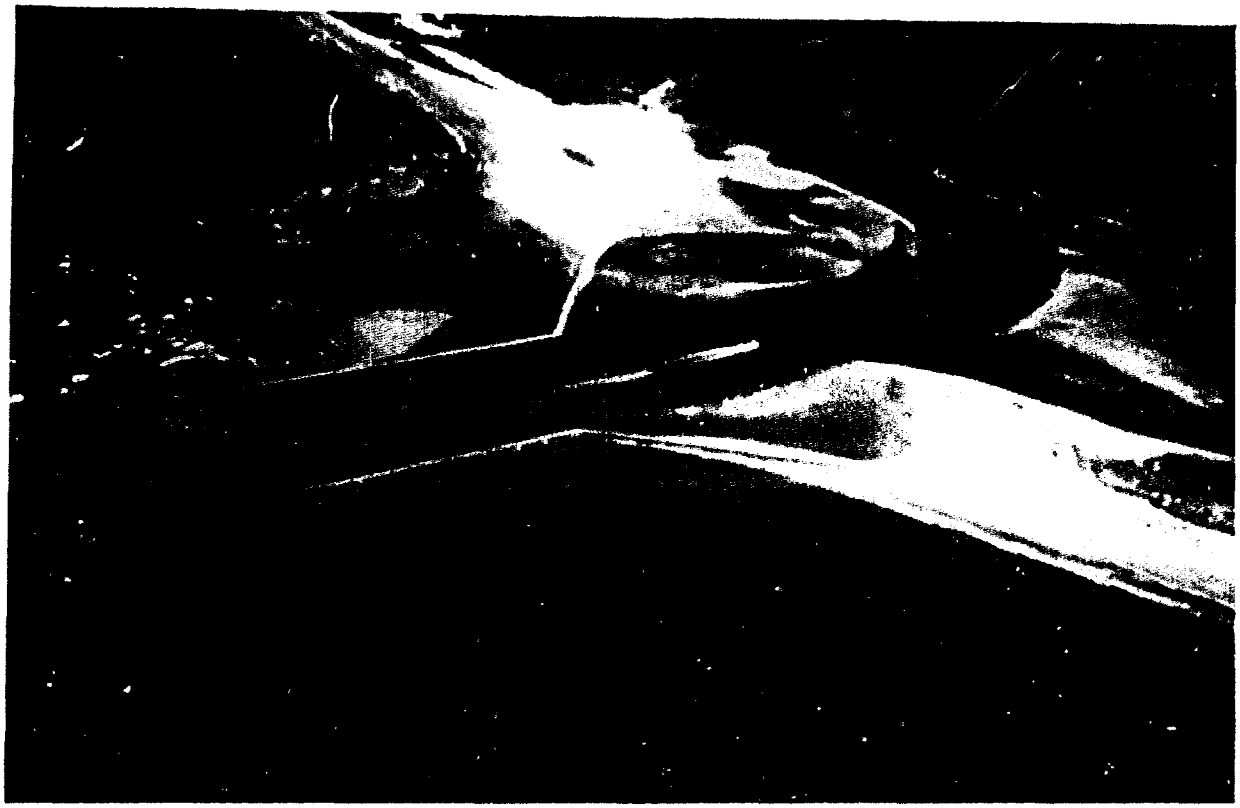

Figure 73. Little River Inlet, South Carolina

273. Prior to construction of the improvements at Little River, a hydraulic model investigation (Seabergh and Lane 1977) was conducted. A review of the study results indicates that the structures built in the prototype were in the same approximate locations as those tested in the model, but some major differences exist between the plan elements constructed and those tested. In the model, 1,300-ft-long stone weir sections (el -2.3-ft mlw) were tested on both the east and west jetties. Also, deposition basins (el -20-ft mlw) were dredged immediately shoreward of each weir section to trap sediment that may move over the weirs due to wave action. As constructed however, the 1,300-ft-long weirs were modified into 650-ft sand tight sections (el +10 ft mlw) on their shoreward ends and 650-ft weirs. In addition, the weirs were subsequently covered with armor stone to an elevation of +8-ft mlw and the deposition basins were never dredged. It also is noted that the interior channel alignment in the prototype is slightly different than that tested in the fixed-bed model.

274. Since the jetties were constructed, the channel has significantly meandered and migrated relative to the constructed project channel alignment.

Scour holes have formed along the inside of the west jetty and at the seaward tip of the east jetty. An engineering assessment of the channel migration and jetty scour problems is currently underway. These problems were not predicted in the fixed-bed model, which examined alignment, length and spacing of the jetties, weir sections, current patterns and magnitudes, effects on the tidal prism, and effects on bay salinities. Stone over the weir sections may have reduced total flows into the basins, which reduce the strong ebb current required to keep the channel in alignment. Since initial construction, dredging has only been required once. This dredged material was placed in the scour hole locations. Sufficient navigable water depths exist in the meandering channel; however, the stability of the channel location and potential impacts to the structures are unknown at this point. An analysis also has been conducted to determine shoreline response as a result of the project. This study determined that the project has not had significant negative impacts on the adjacent shorelines, and the interruption of longshore transport by the jetties has been minimal.

Masonboro Inlet, North Carolina

275. Construction of a jetty (3,639 ft long) on the northern side of Masonboro Inlet was completed in 1965. The jetty entailed an 1,100-ft-long weir section on its shoreward end, and a dredged deposition basin in the lee of the weir section also was included in the project. After construction, however, the navigation channel began migrating toward the jetty which created the potential for scouring and undermining of the structure. In an effort to stabilize the inlet, construction of a 3,450-ft-long south jetty was completed in 1980 (Figure 74). Prior to construction of the south jetty, a hydraulic model investigation (Seabergh 1976) was conducted.

276. A review of the model study results indicated that the structure constructed in the prototype was similar to that recommended in the model. The model test results indicated that the structures should be about the same length to prevent flood tidal currents from swinging toward the north jetty. A south jetty length of 3,400 ft was recommended and a 3,450-ft-long structure was built in the prototype. The model and the prototype channel widths were both 400 ft wide, and the distances between the jetty heads were 1,100 ft for both. The jetty alignment and elevation were also the same in both the model and prototype.

Figure 74. Masonboro Inlet, North Carolina

277. An evaluation of the performance of Masonboro Inlet's entrance since construction of the south jetty reveals that the navigation channel has remained stable between the jetties. Bathymetric surveys subsequent to construction have verified scouring is occurring along the central zone between the jetty structures. Basically, the functional purpose of the dual jetty system has been attained as a result of the south jetty construction. The project, at its entrance, is performing as indicated by the model tests. Through observation, there appears to be some accretion in the channel shoreward of the throat of the inlet that extends to the south. There is a slight build up of sediment in a bend of the channel that could not be predicted in

the fixed-bed model. To date, however, it has not been dredged, nor does it impede navigation.

Port Ontario Harbor, New York

278. Construction of two rubble-mound breakwaters was completed at the mouth of the Salmon River in 1985. The north breakwater was 1,350 ft long, and the south structure was 340 ft long (Figure 75). Before construction of these structures, a sand and cobble bar frequently formed at the mouth of the river, where navigational difficulties were experienced due to the shallow depths and constant shifting of the bar across the entrance. During the peak of the navigation season, when lake levels were normally low, the entrance channel was virtually closed.

Figure 75. Port Ontario Harbor, New York

279. Prior to construction of the breakwaters at Port Ontario Harbor, a hydraulic model investigation was conducted (Bottin 1977b). A review of the model results reveals that the prototype structures were constructed slightly shorter than those recommended in the model study. The model recommended a 1,450-ft-long south breakwater and a 460-ft-long north structure. Test results indicated these structures would stabilize the entrance channel and

provide wave protection to the lower reaches of Salmon River, without adversely affecting the passage of river flows. The prototype south breakwater was constructed on the same alignment as the recommended south structure tested in the model, but it terminated about 100 ft closer to shore. The north breakwater constructed in the prototype was on the same alignment as the model structure but was positioned south of the model breakwater location, which resulted in its reduction in length. In general, the prototype harbor entrance orientation was similar to that tested in the model and the width between the structures were approximately the same, however, this entire entrance configuration was about 100 ft shoreward of that tested.

280. An assessment of the performance of the breakwaters at Port Ontario since construction reveals that it is providing adequate wave protection to the lower reaches of the river as predicted. On the most part, the entrance channel has remained stable with regard to shoaling. In 1989, however, at one point the depth in the entrance channel decreased to about 3 ft. These deposits were believed to be from the river. Because of an extremely dry winter, there was not enough runoff and subsequent river flow to move the river sediment completely into the lake. A rain storm occurred and the depths in the entrance increased from 3 ft to a range of about 7 to 9 ft (8-ft depth authorized). With the exception of this one occurrence, the harbor entrance has remained stable. The structures appear to be performing as predicted with regard to the prevention of channel shoaling due to sediment moving along the shoreline.

Oswego Harbor, New York

281. Oswego Harbor was initially developed in 1882 and consisted of a 4,515-ft-long west breakwater. A 2,700-ft-long west arrowhead and a 2,200-ft-long east arrowhead breakwater were constructed during the period 1931 to 1932. Storm waves propagated through the 650-ft-wide entrance and caused considerable damage to harbor facilities after breakwater construction. To protect the navigation opening and improve wave conditions in the harbor, an 850-ft-long detached rubble-mound breakwater (Figure 76) was completed in 1959. The detached breakwater concept was model tested prior to construction.

282. A review of the model test results of Oswego Harbor (Fortson et al. 1949) indicated that the optimum improvement plan considering wave protection and costs was a 650-ft-long detached breakwater situated 640 ft lakeward of the existing navigation opening. Concern was expressed, however,

Figure 76. Oswego Harbor, New York

that the navigation opening may not be wide enough, and an alternate improvement plan was developed. The alternate plan consisted of the 850-ft-long detached breakwater that was constructed in the prototype. The structure is 910 ft lakeward of the existing navigation opening. This plan should increase navigability of the harbor entrance and provide sufficient protection from waves approaching from critical directions.

283. An examination of the performance of Oswego Harbor since construction of the detached breakwater indicates wave conditions in the harbor are at acceptable levels during storm conditions. No reports of damage or complaints have been received. The harbor appears to be performing as indicated by the model investigation.

Cattaraugus Creek Harbor, New York

284. Construction of two rubble-mound breakwaters was completed at the mouth of Cattaraugus Creek in 1983. The north breakwater was 600-ft-long, and the south structure was 1,850 ft long (Figure 77). Prior to construction of these structures, a sand and gravel bar at the creek entrance, formed by wave induced littoral drift, caused navigation difficulties. Flooding also occurred almost every year due to the presence of the bar at the creek mouth.

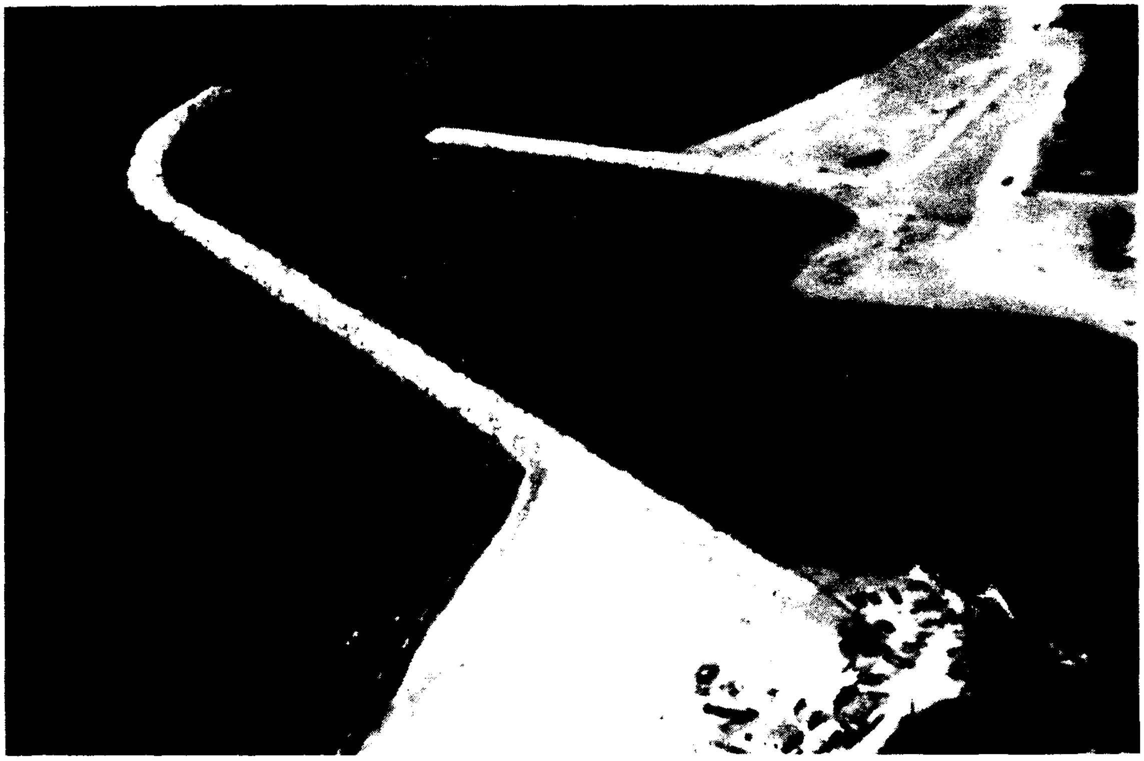

Figure 77. Cattaraugus Creek Harbor, New York

Approximately 400 small boats are permanently based in the lower reaches of the creek.

285. Before the breakwaters were constructed, a model investigation was conducted (Bottin and Chatham 1975). A review of the model report indicates that the prototype structures were constructed almost the same as the recommended model structures. The only difference was that the south breakwater in the model had a dogleg, and the breakwater is curved in the prototype. The ends of the breakwaters were in the same relative locations, and the distance between the heads of the structures is the same as that tested in the model. Therefore, with only the minor change in the trunk of the south breakwater, the results obtained in the model should be applicable to the prototype. The model predicted that the structures would stabilize the entrance channel, provide wave protection to the lower reaches of the creek, and provide for the satisfactory passage of flood and ice flows.

286. The Cattaraugus Creek Harbor project was selected for the MCCP program. Extensive monitoring of the project was accomplished to determine if the completed project performed as intended. Preparation of the report is

currently in progress. The monitoring effort indicates that the model did an excellent job in identifying the best way to eliminate shoaling in the navigation channel; preventing ice jams, recognizing the limitations of the state of the art in modeling lake ice; and designing a channel safe for navigation in high wave conditions. The structures provide wave protection to the lower reaches of the creek, prevent shoaling of the entrance, and allow for the passage of flood and ice flows as predicted by the model investigation. In some instances lake ice has prevented river ice flows from entering the lake and resulting in inland flooding. The use of ice breaking equipment has helped to minimize flooding problems.

Barcelona Harbor, New York

287. Construction of a 693-ft-long east breakwater and a 790-ft-long west breakwater was completed at Barcelona in 1960. The structures were constructed with cellular steel sheetpiling. A 174-ft-long steel sheet-pile shore arm connecting the west breakwater to shore also was included. Subsequent to construction of the breakwaters, a vertical faced public wharf was constructed. Wave energy propagating into the harbor and reflecting off the wharf and vertical cellular breakwaters resulted in standing and multidirectional waves up to 4 ft in height. In 1984, construction of a 250-ft-long, lakeward west breakwater extension, a 150-ft-long shoreward east breakwater extension, and segmented wave absorbers adjacent to the harbor side of the west breakwater was completed (Figure 78). The modifications were constructed of rubble-mound materials. The 1984 modifications were tested in a model prior to construction.

288. A review of the model investigation of Barcelona Harbor (Bottin 1984a) indicated that the harbor modifications were constructed in the prototype as recommended by the model test results. The harbor revisions should reduce wave heights in the mooring areas to 2 ft or less during periods of storm wave activity with the exception of summer waves that have a 20-year recurrence interval. Wave heights for 20-year summer conditions will exceed the 2-ft criterion by 0.2 to 0.3 ft for waves from the west and northeast. A parapet wall on the west breakwater and a shoreward extension of the east structure would achieve the desired criterion from these directions, but were not constructed in the prototype.

289. An examination of the performance of Barcelona Harbor since construction of the 1984 modifications reveals that the harbor is functioning

Figure 78. Barcelona Harbor, New York

well during periods of storm wave activity. After construction, in December 1985, a record storm hit the harbor. It performed well with no reports of damage. Local harbor users are well pleased with the project. What was almost an unusable harbor, before improvements, is now highly utilized. In fact, the harbor is at capacity with no room for expansion. The project is performing as predicted by the model study.

Conneaut Harbor, Ohio

290. Construction of east and west jetties was completed at the mouth of the Conneaut River in 1894. During the period 1905 to 1936, outer east and west breakwaters were constructed lakeward of the river mouth. Hydraulic problems encountered at the harbor after this construction included excessive wave heights in the outer navigation entrance, and navigational problems in the inner entrance due to the alignment of the jetties with excessive seiche currents across the entrance. In 1965, construction of a shore-connected east breakwater was completed and the east jetty was realigned and shortened (Figure 79). The 1965 improvements were model tested prior to construction.

291. A review of the model investigation (Hudson and Wilson 1963) indicated that an extension of the east breakwater to shore would reduce currents

Figure 79. Conneaut Harbor, Ohio

at the entrance to the inner harbor to acceptable levels. The study also recommended removing or shortening and realigning the east jetty parallel to the west jetty to provide a satisfactory navigation width without permitting increased wave action in the inner harbor. In addition, the model investigation recommended a detached 900-ft-long breakwater situated about 1,600 ft lakeward of the outer navigation opening to provide additional wave protection to the outer entrance and inner harbor. The east breakwater shoreward extension constructed in the prototype was the design recommended in the model. A 100-ft-wide gap between the existing breakwater and extension provided the passage of light-draft vessels without affecting current velocities at the inner harbor entrance. The shortening and realigning of the east pier performed in the prototype also was recommended in the model to improve navigability. The offshore detached breakwater recommended in the model to reduce wave heights in the outer entrance inner harbor, however, was not constructed in the prototype.

292. An examination of the performance of the harbor after the 1965 improvements reveals improved navigability into the inner harbor as predicted

by the model study. The shoreward extension of the east breakwater eliminated excessive currents across the entrance to the inner harbor as expected, and shortening and realignment of the east jetty provided a satisfactory navigation width. No navigation problems have been reported since construction. An erosion problem, however, was encountered along the shoreline east of and adjacent to the east breakwater extension after construction of the structure. The erosion, due to wave reflection off the vertical-wall breakwater, extends eastward about 1,000 to 1,500 ft. The area is undeveloped and the shoreline appears to have reached equilibrium, therefore, no remedial action has been taken.

Geneva-on-the-Lake Small-Boat Harbor, Ohio

293. Construction of Geneva-on-the-Lake Small-Boat Harbor was completed in 1987. Included in construction were two rubble-mound breakwaters. The east structure was 600 ft long and the west breakwater was 850 ft long. The structures originated as an arrowhead configuration converging toward each other, but the outer 200 ft of the breakwaters are parallel (Figure 80). Spurs, about 25 ft long, extend lakeward from a revetted entrance. A

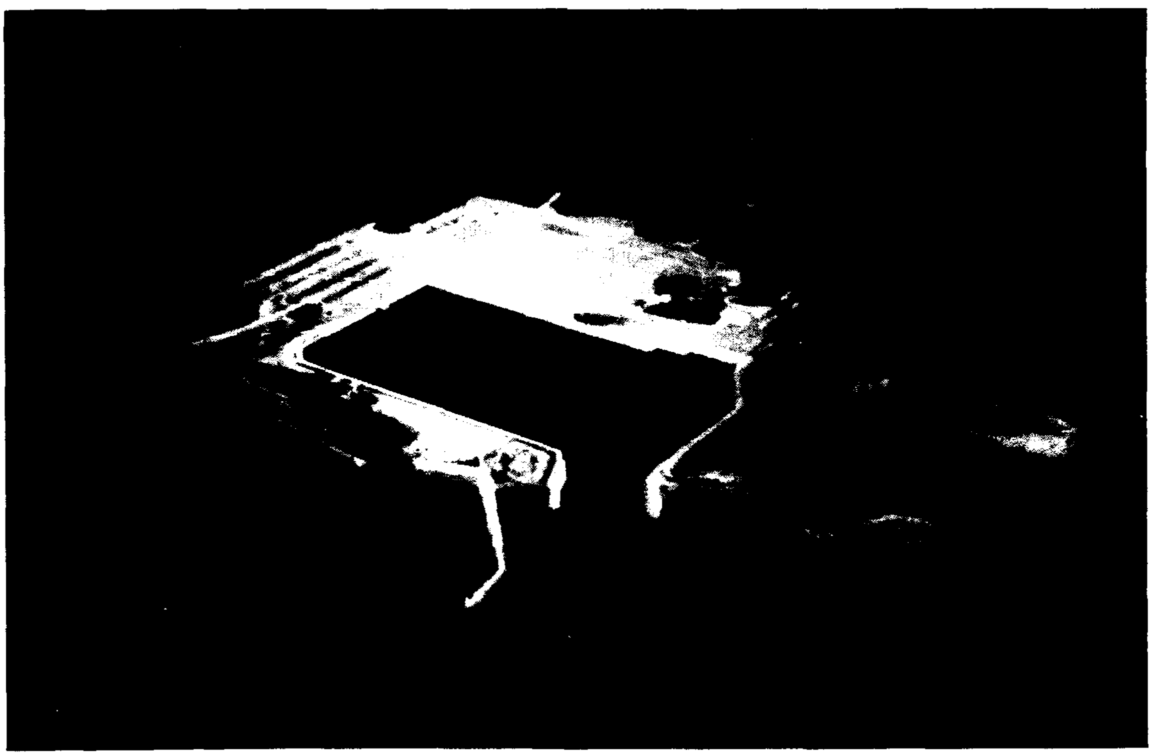

Figure 80. Geneva-on-the-Lake Small-Boat Harbor, Ohio

revetment also extends along the west and south sides of the harbor. The harbor was constructed to provide a small-boat harbor-of-refuge and recreational facilities for boaters in the area. It borders Geneva State Park.

294. Prior to construction of the small-boat harbor, a model investigation was conducted (Bottin 1982a). A review of the model report indicates that the harbor was constructed in accordance with recommendations of the model investigation. Test results indicated this configuration would provide adequate wave protection to the harbor and prevent shoaling of the entrance channel.

295. A review of the postconstruction performance of the small-boat harbor reveals that no complaints have been received relative to wave conditions in the entrance and mooring areas and/or entrance shoaling. The prototype harbor is performing as predicted by the model investigation. The breakwater structures are trapping sediment moving from west to east, however, and beach replenishment, as well as other protective measures, have been implemented on the east side of the harbor. This condition was expected since the model study indicated no natural sand bypassing at the site.

Vermilion Harbor, Ohio

296. Construction of parallel jetties at the mouth of the Vermilion River was completed during the period 1836 to 1839. In 1874 the east and west jetties were extended to lengths of 458.5 ft and 1,333.5 ft, respectively. Storm waves, however, would break inside and immediately outside the entrance jetties, which made navigation difficult and dangerous, even during moderate storms. In 1973, an 864-ft-long detached breakwater was completed at Vermilion (Figure 81) to provide wave protection to the entrance. It was constructed with cellular steel sheetpiling. The offshore breakwater concept was model tested.

297. A review of the model study report (Brasfeild 1970) revealed that a 700-ft-long offshore breakwater situated perpendicular to the entrance channel about 200 ft from the outer end of the existing east jetty would provide adequate wave protection. The offshore structure built in the prototype was 164 ft longer than that recommended in the model. In addition, it was located 330 ft from the lakeward end of the east jetty. The model study report did, however, address the possibility of a longer structure placed further lakeward to provide more navigation clearance. Even though model tests were not conducted, and it could not be stated categorically, it was believed that the

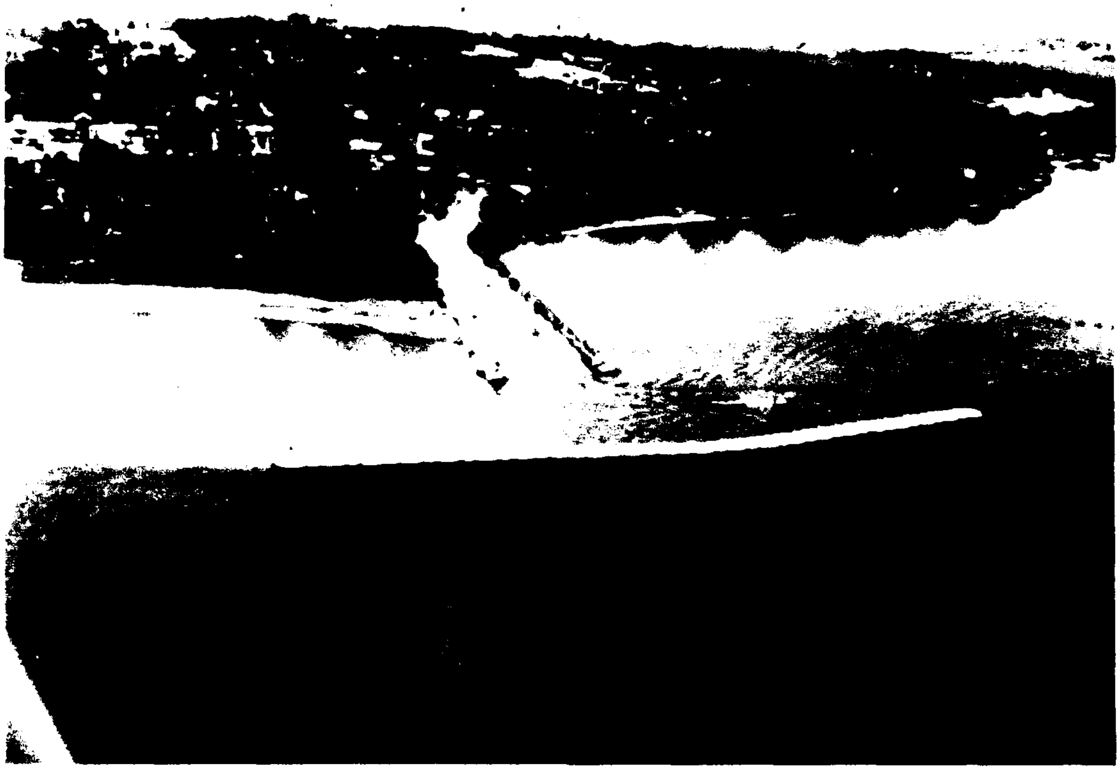

Figure 81. Vermilion Harbor, Ohio

longer, more lakeward structure would perform as well as the 700-ft-long recommended breakwater. The overlap of the existing entrance for waves from the extreme easterly and westerly directions was similar.

298. A review of the performance of the detached breakwater indicates that the structure is very effective for storm wave reduction. No complaints have been received relative to undesirable conditions in the mooring areas of the harbor. Once since construction, in the early 1980's, complaints were received stating that the detached structure was constricting ice flow from the Vermilion River and contributing to an ice jam. The structures are far enough apart, however, to allow Coast Guard ice cutters access to ease jamming if this problem occurs in the future. Relative to wave conditions, the breakwater is performing as predicted by similar plans studied during the model investigation.

<u>Ludington Harbor, Michigan</u>

299. Construction of north and south jetties at the entrance to Pere

Marquette Lake at Ludington was completed during the 1866-1874 time frame. Later the construction of arrowhead breakwaters forming an outer harbor at the site was completed during the 1907-1914 period (Figure 82). Periodic repairs and maintenance have been performed on the structures since construction. From 1977 to 1981, the south jetty was reconstructed using rubble-mound structures, and a rubble absorber was installed adjacent to the north jetty. Several improvement plans at the inner entrance were model tested prior to the construction.

Figure 82. Ludington Harbor, Michigan

300. A review of model test results (Crosby and Chatham 1975) indicates that a 500-ft-long rubble absorber on the south side and a 900-ft-long rubble absorber on the north side of the channel would provide the desired level of protection with respect to wave action in the inner harbor and navigation conditions in the inner entrance. The lengths of the rubble absorbers installed in the prototype were 505 and 1,061 ft for the south and north jetties, respectively.

301. A review of the improvements at Ludington Harbor as a result of the model study reveals that the absorbers perform quite satisfactorily. Wave

energy is attenuated as waves propagate down the channel. Small-craft mooring at an area near the inner end of the rubble absorbers is satisfactory, and entrance conditions between the structures have improved. Conditions are definitely suitable and are similar to those predicted in the model study.

New Buffalo Harbor, Michigan

302. Construction of two breakwaters totaling 2,045 ft in length was completed in 1975 at the mouth of the Galien River (Figure 83). The breakwaters, with the exception of a 200-ft section of the north structure, were constructed of rubble-mound materials. The 200-ft-long test section was a frame-type breakwater constructed of z-shaped steelpiling and supported by H-beams. The slope of the lakeward face of the structure was 1v:2h. Prior to construction of the breakwaters, mooring facilities upstream were exposed to damaging wave action, and a significant shoaling problem existed at the river mouth. Several breakwater configurations were model tested before construction.

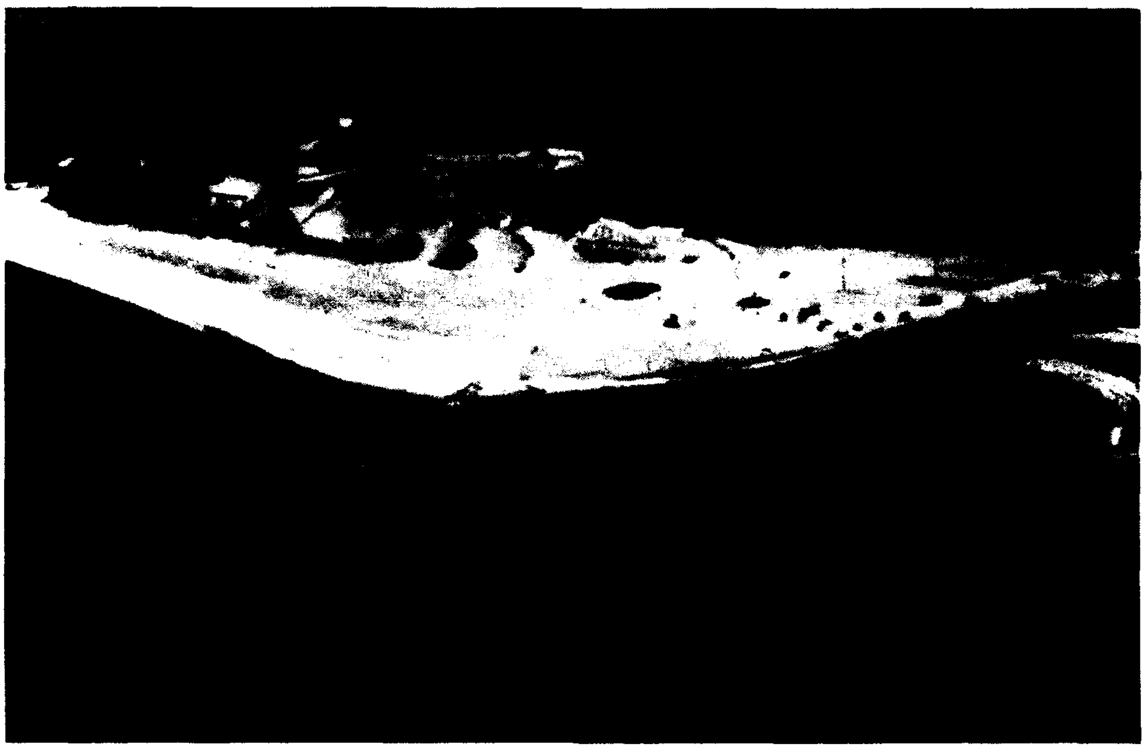

Figure 83. New Buffalo Harbor, Michigan

303. A review of the model test results (Dai and Wilson 1967) of New Buffalo Harbor reveals that the 1,305-ft-long north breakwater constructed in the prototype was the same as that recommended in the model investigation, with the exception of the test section. Frame-type breakwaters were tested in the model, in place of the rubble structures, but were not recommended due to significant overtopping, even though they were constructed with crest elevations 2 ft higher than the stone structures. The wave height criteria could not be achieved with the frame-type structures. The south breakwater constructed in the prototype was approximately 120 ft shorter than that tested and recommended in the model. Both the model and prototype south structures were oriented to provide a 200-ft-wide navigation entrance. The prototype south breakwater originated closer to the north structure which resulted in the length reduction. The breakwaters constructed in the prototype should probably provide greater protection than those model tested since the south breakwater was overlapped more by the north structure. The model breakwaters predicted that wave heights in the entrance would not exceed 2.5 ft and wave heights in the inner basin would not exceed 1.5 ft.

304. An assessment of the performance of the New Buffalo Harbor breakwaters reveals that the mooring areas are well sheltered during storm wave conditions and no complaints have been reported. The frame-type portion of the north breakwater is sometimes overtopped, but this does not appear to be a significant problem. As expected, sediment does accumulate against the north breakwater with some migrating into the channel. Also, the beach is periodically nourished on the south side of the structures. The structures at the site are functioning well and conditions exist as predicted by the model investigation. Overall, the project is considered very successful.

Port Washington Harbor, Wisconsin

305. Construction of a 2,537-ft-long north breakwater and a 1,006-ft-long south breakwater was completed during the 1934-36 time frame. Also included was the construction of two inner slips with vertical steel sheet-pile walls. In 1940 a north pier was constructed at the entrance to the inner harbor slips. After construction, waves from northeast clockwise through south-southeast occasionally caused considerable damage to harbor facilities. Waves passed through the navigation opening and propagated into the slip area where serious problems occurred due to wave reflection off the vertical walls. Also wave overtopping of the north caisson breakwater generated hazardous

conditions in the outer harbor area. In 1982 rubble-mound breakwaters, an absorber, and a parapet wall were conducted to develop a small-craft harbor in the existing Port Washington outer harbor (Figure 84). Prior to construction of the small-craft harbor, improvements were model tested.

306. A review of the model investigation of the small-boat harbor (Bottin 1977a) revealed that the structures built in the prototype were the same as recommended in the model investigation. Improvements consisted of a parapet constructed on a portion of the existing north breakwater in conjunction with an absorber installed on the shoreward side and adjacent to that portion of the breakwater, and new east and west rubble-mound breakwaters enclosing the new harbor. The east breakwater was attached to the existing north breakwater and the west breakwater was detached to provide for increased circulation in the harbor as recommended in the model study. Also recommended in the model, and constructed in the prototype, was steel sheetpiling in the west breakwater to make the structure impervious, thus preventing transmitted wave energy from entering the harbor.

Figure 84. Port Washington Harbor, Wisconsin

307. Immediately after construction of the new small-boat harbor at Port Washington, it was subjected to a storm and performed very well. The Ozaukee Press published an article entitled, "The harbor is finished . . . and it works." The article stated that . . . "foul fall weather with strong east-northeasterly winds put Port Washington's new small-boat harbor to the test -- and it passed with flying colors. The harbor was an oasis of calm assaulted ineffectually by rough seas on three sides. Waves crashing into the old breakwater were robbed of their power by the new concrete parapet. The two new overlapping stone breakwaters repelled waves coming through the wide open outer harbor gap." The article also stated that the storm showed off the benefits of the new small-boat harbor and was heartening news to the community and boat owners -- the boats moored there will have a safe haven. Subsequent storms at Port Washington Harbor have also revealed very calm conditions in the new harbor. Since construction of the improvements, there have been no negative reports relative to the small-boat harbor's performance, and it has reacted as predicted in the model investigation. The harbor is filled to capacity with small craft.

Grand Marais Harbor, Minnesota

308. Breakwaters were constructed and/or modified at the entrance to Grand Marais Harbor intermittently between the period 1883 to 1933. After construction of these structures, however, storm waves continued to propagate through the entrance into the harbor and cause damage to small boats. Small craft had to be beached to prevent damage, even for mild storms. In 1959, construction of a 921-ft-long rubble-mound inner breakwater was completed to provide protection for small craft in its lee (Figure 85). The inner breakwater had been model tested.

309. A review of the model investigation of Grand Marais Harbor (Fenwick, Arnold, Easterby 1944) indicated that the inner breakwater constructed in the prototype was similar to the configuration tested, but some discrepancies did exist. The model structure entailed a 600-ft-long timber crib breakwater that provided protection to the proposed mooring area with a 290-ft-long rubble-mound portion extending from the western end of the timber crib at an angle to the shoreline. The model report recommended the construction of a rubble-mound structure, as opposed to the timber crib, because wave conditions tested behind the rubble section were less confused and lower in height. A straight 921-ft-long rubble-mound structure was constructed in the

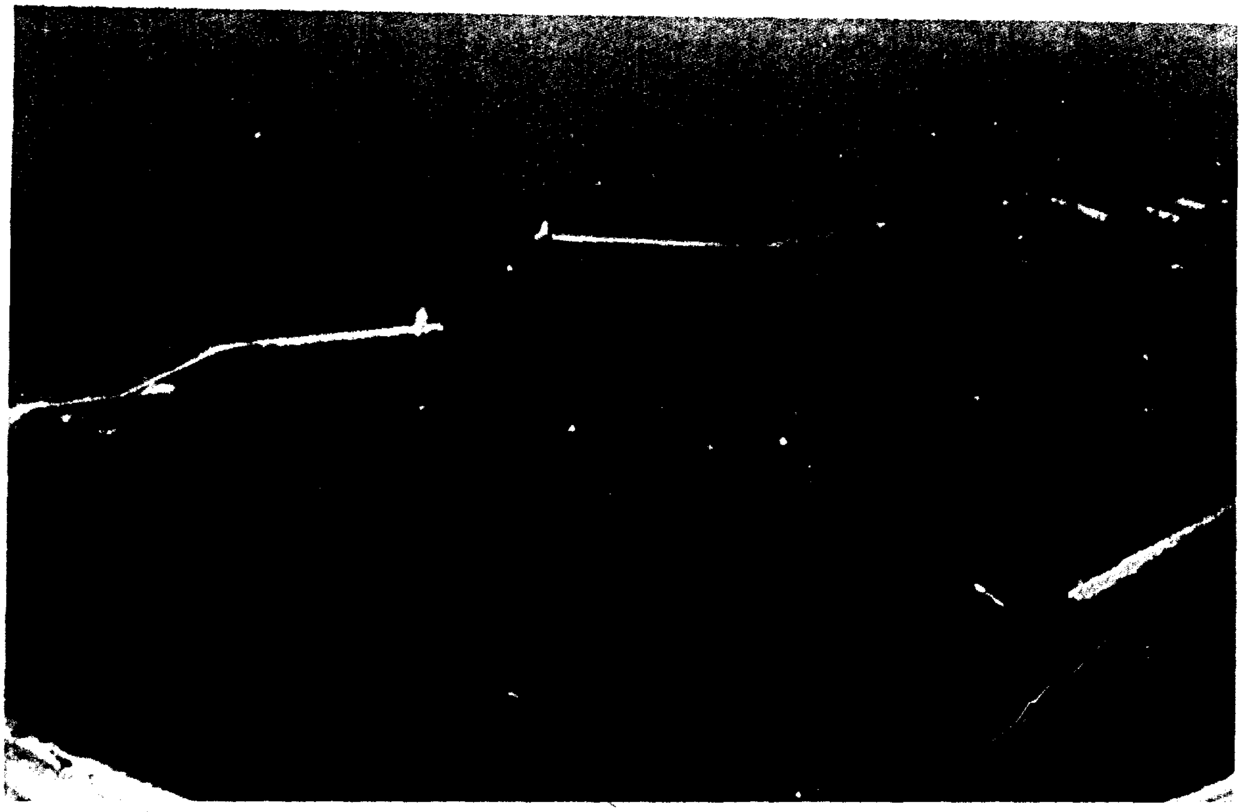

Figure 85. Grand Marais Harbor, Minnesota

prototype that provided protection to the mooring area and the north bank. The lakeward end of the inner breakwater, however, terminated about 100 ft westward of that tested in the model, not providing quite as much overlap for the harbor facilities.

310. A review of the performance of the inner breakwater reveals that it provides adequate wave protection to small-craft vessels and harbor facilities in its lee. The breakwater does attenuate large waves that propagate through the outer entrance. The harbor receives heavy usage and is said to be cramped. The depths in the harbor are too deep for economical extension of the breakwater and expansion of the small-boat harbor. Wave conditions behind the Grand Marais Harbor inner breakwater, however, appear to be within the range predicted by the model study for storm wave conditions.

Discussions

311. Twenty-five small-boat harbors have been constructed in the prototype subsequent to being model tested at WES. All these harbors have been

reviewed to determine, first, if they were constructed as recommended in the model investigations and, secondly, to determine if they have performed as predicted by the model studies. The performances of the projects were obtained from reliable sources, but were not detailed and indepth in most cases. The data obtained gives an indication of how the projects have performed and includes any problems encountered since construction.

312. Of the 25 small-boat harbors that have been constructed, nine projects were constructed exactly as recommended in the model investigation. An additional 10 projects were constructed with slight modifications. In some cases, these projects consisted of structures that were curved instead of having a dog-leg, slightly longer structures than recommended, and/or a reorientation that would provide the same overlap and should provide equal protection. These 10 projects appear to be providing the same relative protection to the harbor as the recommended plans developed from the model studies. The other six harbor projects were constructed with the same basic configurations as recommended in the models, but they had shorter structures, wider entrances, deeper depths, and/or may have omitted an element (or structure) of the design. Variations from the recommended design of these six projects could reduce their functional efficiency.

313. An assessment of the performance of the prototype harbors reveals that most perform as predicted by the model studies. Three of the projects receive complaints of excessive wave action in the mooring area and/or at a boat ramp. A review of the model investigations, however, indicated that the established wave height criteria were 2.0 ft or greater at these locations for storm wave conditions. For the more current model studies, a criteria of 1.0 ft or less is generally established. Model tests for the three studies in question show wave heights of 1.0 to 1.6 ft during everyday wave conditions. Model studies predicted the wave conditions correctly; it is just that the selected wave height criteria for the studies was too high. Other projects have performed very well during storm wave conditions. Tracer tests were conducted in the models of several projects to qualitatively assess sediment deposition and scour patterns. All these prototype projects have been evaluated with regard to shoaling, and it appears they are performing as predicted by the model test results. Shoaling of the project entrances or mooring areas has not occurred since construction. Structures have been constructed in the prototype for three tidal inlet sites that were model tested.

The entrance channels are stable at two of the prototype sites, but at the other site, the channel has migrated and meandered between the jetties (causes of the channel migration are currently under investigation). Also, the model was a fixed-bed study, which makes it extremely difficult to predict a stable channel. In summary, with the exception of the one tidal inlet study, the projects constructed in the prototype, that did not vary radically from the designs recommended for the model studies, are performing as predicted by the model investigations.

PART IV: SUMMARY OF LESSONS LEARNED

314. The interaction of storm wave phenomena at harbor sites (propagation of wave energy into harbors, diffraction of energy through harbor entrances, reflection of energy from structures and facilities, energy reaching inner harbor areas through waves overtopping and/or being transmitted through protective structures, wave generated currents, and/or storm surge currents, etc.) creates very complex hydrodynamic conditions. When river and/or tidal currents are present, the complexity of the dynamic hydraulic system increases. Also, shoals formed in the entrances may cause breaking waves or may redirect wave energy, through wave refraction, to other areas in the harbor (areas that generally may not experience problems). Through the use of physical modeling, many lessons have been learned, however, optimizing the design of a project is still a difficult problem which requires the accurate use of the best design tools available. The physical model has proven itself a reliable and economically justified tool.

315. The first step for successful design of a harbor project is to define up-to-date, realistic design conditions as well as up-to-date bathymetric and topographic data. The designer should develop a data set defining storm wave characteristics (period, height, direction, spectral shape, etc.) at the site. He/she should ensure that tidal conditions (tidal heights, velocities, prism, etc.), river discharges, ice problems, predominant direction of sediment movement, as well as volumes, long-period wave conditions, and seiche activity, if applicable, are well defined or will be determined through the course of the study. Design of a project is very dependent upon high-quality selection and definition of hydrodynamic design conditions. The designer should also establish a reasonable performance criterion for the project that ensures that the design will function satisfactorily relative to the needs of local harbor users. Prototype performance of various harbors indicates that acceptable wave height performance criterion was met at several sites but was too high at other sites, and continual complaints by harbor users have been received since project construction.

316. Studies have shown that it is generally desirable to prevent wave energy from entering a harbor as opposed to attempting to dissipate the energy once inside the harbor. Wave energy entering a harbor can be minimized by overlapping breakwaters at the entrance, reducing the entrance width if

possible, constructing breakwaters that are not easily overtopped by waves, and using impermeable breakwater cores to the extent possible to reduce wave transmission. Offshore structures, constructed seaward of the entrance may also be incorporated into a design. Care must be taken, however, if the harbor is at a river mouth. Construction of structures at the entrance opening may contribute to flooding upstream, due to flow restriction and/or ice jamming in the colder climates.

317. Physical limitations or project costs may prohibit the construction of structures which will allow less wave energy into a harbor. Studies reveal that it is very difficult to reduce wave energy once it enters the harbor; however, measures can be taken to absorb some of this energy. Rubble wave absorbers and/or spending beaches may be constructed at critical locations (where wave energy is high) in the harbor and effectively absorb energy and, thus, reduce undesirable wave effects. Also, in areas where standing waves exist, spending beaches, rubble absorbers, and/or concrete absorber units (i.e. such as igloos) may be effective in the reduction of wave energy.

318. Harbor entrance channels should not be aligned parallel to incoming wave crests. In this event, small craft must enter broadside to incoming waves which is very hazardous. In areas where these conditions currently exist, breakwater arms or extensions may be constructed seaward and parallel to the entrance channel. This will provide a calm area in which vessels can be controlled prior to entering the harbor.

319. Vertical-wall breakwaters and harbor structures are highly reflective and should be avoided in areas where higher levels of wave energy are present. Waves reflected off entrance structures result in very confused and hazardous conditions for navigation. Vertical walls inside harbors where wave energy is present cause hazardous anchorage and mooring conditions resulting in more frequent damage to facilities and small boats. Reflected waves from vertical structures also induce erosion.

320. Model tests have shown that absorbers installed along the slips of various harbors are essentially ineffective in dissipating long-period wave energy. In harbors where the mode of oscillation of a basin is excited by the frequencies of wave energy entering the harbor, the basins will respond and standing waves may result which could result in damages to both moored vessels and dock facilities. Changing the geometric configuration of the basin could

remedy the condition, but the new configuration may respond to another wave frequency. These problems are very difficult to alleviate, however, model testing has indicated that long-period wave energy entering a harbor may be reduced by a structure that overlaps the entrance opening. An offshore structure seaward of the entrance also may reduce oscillations in the harbor basins. To be effective, the offshore structure needs to be relatively impermeable.

321. Harbor facilities and/or boat ramps should not be constructed adjacent to an entrance opening. Wave energy propagating through and diffracting around entrance structures may impinge upon the facilities and cause undesirable conditions. In areas where physical limitations exist, harbor facilities may be protected by interior breakwater structures or revetted moles. Prototype performance of various harbor sites has shown these improvements to be very effective, however, when inner structures or moles are constructed, future expansion is sometimes limited. Inner breakwaters and revetted moles also have been used to protect harbor facilities in the lee of existing structures where overtopping and/or excessive wave transmission exist. In other instances, breakwaters have been raised and sealed, or parapets have been installed to minimize the amount of wave energy entering the harbor.

322. Harbor entrances should not be oriented toward the upcoast side in which sediment is moving alongshore. In most cases, the protective structures will serve as a trap and sediment may deposit in the entrance channel. It is very desirable to build structures that will contribute to natural sand bypassing of the harbor entrance. Tests have shown that structure and entrance openings oriented toward the downcoast side of the predominant direction of sediment movement contribute to natural bypassing. An outer curved breakwater which overlaps a short shore-connected structure allows sediment to move around the entrance and on downcoast. The shorter downcoast structure assists in minimizing movement of sediment along the shoreline, due to eddies and periods of reversal of sediment movement, and into the entrance. Prototype performance of several harbor sites have validated the qualitative methods of determining sediment movement used in model investigations as reliable tools. Improvement plans were developed that would prevent shoaling in the harbor entrances and/or mooring areas. These plans, after being constructed in the prototype, proved very effective.

323. At some existing harbors, sediment moving alongshore is intercepted by a structure and moves along its axis where it deposits in the entrance channel. Model tests and prototype performance have indicated that spurs installed adjacent to the seaward sides of the structures tend to deflect flow (sediment) away from the entrance. Spurs, in some model tests, also have been used to deflect undesirable crosscurrents away from harbor entrances.

324. Model tests and subsequent prototype performance have shown that segmented breakwaters are effective in providing wave protection while still allowing tidal circulation through the breakwater openings. They proved more effective than baffled breakwaters in model tests. Consideration may be given to using segmented structures in lieu of floating or baffled breakwaters, where both wave protection and harbor flushing is required. Segmented rubble absorbers inside a harbor, as opposed to a continuous absorber, also have been effective in both the model and in the prototype. This alternative may result in significant cost savings through reduction of stone quantities.

325. Waves breaking across reefs generally result in very strong currents alongshore. These crosscurrents may be hazardous to small craft entering and navigating channels cut through the reef and into harbors. Model tests and subsequent prototype performance have indicated that breakwaters may be used to deflect these currents offshore away from the entrance, and thus, alleviate or minimize hazardous crosscurrents. Circulation channels have also been very effective in providing harbor flushing for harbors situated in reef areas.

326. In tidal inlets, an even distribution of flow (without excessively high or undesirably low velocities) is required. Model tests indicate that the jetties used to stabilize the inlet should in most cases be of equal length. They should also be parallel or dikes should be installed to divert or concentrate the flow of tidal currents through the desired channel alignment and thus prevent channel meandering and possible undercutting of structural toes. The width between the jetties is important to a stable inlet and depends upon the volume included in the tidal prism or the actual tidal flow exchange through the inlet. Where weir sections and deposition basins are used to trap sediment moving alongshore, it is important that location of the weir be properly selected. The weir should also be installed low enough to

allow adequate velocities and sediment to move over the structure and into the deposition basin but high enough to prevent excess wave agitation.

PART V: FUTURE RESEARCH ACTIVITIES RECOMMENDED

327. Complete design guidance on conditions required to provide safe mooring in small-boat harbors has not yet been developed. For the hydraulic model investigations conducted at WES, wave height criteria established in the harbor mooring areas have ranged from 0.5 to 2.0 ft. The sponsor of each model investigation tends to establish a rule-of-thumb wave height criterion for small-boat harbor mooring. Research should be conducted to establish Corps of Engineers design guidance on harbor mooring criteria for small-craft vessels of various sizes (i.e. fishing craft, small commercial craft, and recreational boats) as well as for various types of docks and mooring systems. Prototype wave data could be obtained to determine conditions which cause damage and/or produce problems or hazardous mooring conditions. These data, among other considerations, could be used as a basis of establishing the criteria.

328. Navigation of small-craft vessels through a harbor entrance in a severe wave environment can be very hazardous; however, adequate design criteria have not been established with regard to safe navigation conditions. Entrance channel wave height criteria varied from 3 to 6 ft in hydraulic model investigations conducted at WES. Research is needed to establish safe navigation conditions for various-size vessels and entrance types. Prototype data should be obtained to establish what level of wave and/or current conditions produce unsafe entrance conditions for various-size vessels. In addition to wave characteristics, instrumentation could be installed on small craft to measure vessel response when conditions are approaching an unsafe level. Vessel response also could be determined by use of scale model small craft with scaled wave conditions of the prototype in a physical model. Model vessel response relative to a wide range of wave conditions and directions could be defined to determine entrance channel criteria for safe navigation.

329. In many harbors, it is not feasible to construct offshore breakwaters to prevent wave energy from entering the harbor. Excessive wave energy and/or standing waves, therefore, exist within the harbor during storm events. Research should be conducted to determine the most effective and efficient placement of spending beaches or wave absorbers in harbors and/or harbor basins. Slopes of the beaches and absorbers, as well as their placement, along with development of various types of wave absorbers could be investigated in

physical models. Test results for rubble-wave absorbers with various slopes and stone sizes could be compared with concrete absorber units (i.e. such as igloos), as well as new concepts, to determine the optimum absorber design for various design wave conditions and harbor geometries.

330. Shoaling of harbor entrances is a major problem and causes difficult and hazardous navigation conditions. In addition, expensive maintenance dredging or artificial bypassing is required. Model tracer tests for site specific projects have indicated that design features of the entrance structures can be improved to minimize or alleviate channel shoaling. Model tests also have revealed that spurs installed adjacent to structures can deflect flow (sediment) away from the entrance channel. Research should be conducted to determine an optimum design for the layout of harbor entrance structures with regard to channel shoaling. Based on model tests, the predominant mode of sediment movement is in the breaker and swash zones along the shoreline. The angle of a structure relative to the shoreline and relative the angle of wave approach at the structure appears to be critical. It has been noted that reflections from some structures prevent sediment accumulations in the entrance channel. The angle of deflection of longshore currents by structures, or spurs, also appears, in some cases, to deflect sediments into deeper water seaward of the breaker zone. Research also should be conducted to determine an optimum design for the natural bypassing of sediments around harbor entrances.

331. Many harbors that provide adequate storm wave protection experience problems with harbor circulation or flushing. Stagnation causes undesirable water quality. Research should be conducted to improve and/or develop methods for providing improved harbor flushing. The proper placement of structures that would redirect wave-generated currents and/or tidal currents should be studied. Also, the effectiveness of circulation channels and/or gaps in breakwater structures or baffled and segmented structures, that would provide for circulation and not increase wave conditions in the harbor, should be examined.

332. Many harbors on the ocean coasts experience long-period wave problems which result in significant horizontal water movements in nodal areas and vertical motions at antinodes of the oscillation. This occurs when the mode of oscillation of the harbor basin is excited by frequencies of incident wave energy entering the harbor. Vessels and harbor facilities are frequently

damaged when these conditions occur. Minor structural modifications are ineffective in alleviating these undesirable conditions. Research should be conducted to determine the most effective solution to this problem. For some cases that have been model tested, relatively impervious structures seaward of the entrance show some promise in reducing this wave energy. This alternative should be subjected to more extensive testing. In addition, other alternatives should be examined. Changes in geometric shapes of harbors (i.e. rounded basins versus rectangular basins) and the installation of flat slopes and/or solid barriers in harbor basins to change the oscillations patterns should be investigated.

333. Numerous problems occur in tidal inlets along the coasts of the United States and it territories. The interaction of wave-induced currents and tidal currents, sometimes with freshwater discharges, through an inlet connecting the ocean with an embayment is very complex. Extensive dredging is required to maintain adequate depths in navigation channels, and maintenance of jetties at inlet openings can be a serious problem due to continued scour and undermining of structures. An undistorted physical inlet model should be constructed and tested with a range of wave conditions superimposed on simulated tidal conditions. Waves and currents throughout the inlet could be quantified and a better understanding of the fundamental hydrodynamic processes could be realized. Tracer tests could be conducted in the inlet to give qualitative indicators of sediment patterns resulting from wave-current interactions. Research then could be conducted with movable-bed areas in an effort to quantify sediment movement as a result of various test conditions. After the fundamental processes are better understood, research should be conducted to develop methods to better stabilize inlet openings, minimize shoaling, and reduce maintenance costs.

PART VI: PHYSICAL MODELING GUIDELINES

334. This part of the report provides guidelines that designers of small-boat harbors can use to determine when and what type physical model investigation can be used to enhance and optimize the project design. Small-boat harbor models, in general, are discussed as well as design information required for a physical model investigation.

Small-Scale Harbor Models

335. Three-dimensional harbor wave action model studies have been conducted at WES since the 1940's. The hydraulic scale model is commonly used as a design tool to aid in the planning of harbor development and in the design and layout of breakwaters, jetties, groins, absorbers, etc. to obtain optimum harbor protection and verify and/or define suitable project performance. Modeling techniques and procedures as well as methods of data collection have been maintained at the leading edge of technology, and reproduction in the model of very complicated hydrodynamic phenomena has become possible through experience gained during the conduct of reimbursible work and basic and applied research. The small-scale physical hydraulic model is the most reliable means of determining an optimum plan for harbor or inlet improvements, particularly when short-period wave effects are prevalent. The model can reproduce breaking waves, wave-current interactions, and the simultaneous effects of wave refraction, diffraction, overtopping, transmission, and reflection.

336. Small-boat harbor design is very difficult due to the complexity of hydrodynamics and geometry of most harbors. Physical hydraulic models generally are used to:

a. Determine the optimum location, alignment, height, length, and type of breakwaters required to provide adequate wave protection in harbor mooring areas and entrance channels and to quantify wave and current characteristics.

b. Determine the location, alignment, and composition of spending beaches and/or other energy dissipating structures inside the harbor area. These may be rubble-wave absorbers or concrete absorber units.

c. Determine the optimum length and/or alignment of breakwater or jetty structures required to minimize shoaling in harbor entrance channels. Information on the effects of structures on the littoral processes can be gained through the use of model tracer materials.

d. Determine the optimum length and/or alignment of breakwaters, jetties, or spurs required to alleviate undesirable wave-induced crosscurrents and/or shoaling in harbor entrance channels. Plans to provide for increase wave-induced harbor circulation and/or flushing also may be optimized through studies and quantification of current velocities developed by various design alternatives.

e. Determine the response of various harbor configurations or expansions to long-period wave energy incident to the site. Nodal and antinodal areas may be identified in the harbor for various frequencies and the water velocities and water surface motions can be quantified at these locations.

f. Determine river flood and/or ice flow conditions that may enter in or adjacent to a harbor, and quantify the impact of that structural modification at the harbor will have on these conditions. In addition, qualitative information on bed load river sediment movement may be defined.

g. Study and quantify the effects of harbor modifications on thermal stratification through the use of heated and dyed water input at designated locations.

h. Determine tidal currents or seiche generated currents in a harbor or its entrance. Tidal currents through an inlet entrance may be studied and quantified to determine the optimum placement of structures required to stabilize the inlet and minimize maintenance dredging.

337. During the conduct of model investigations, remedial plans are developed, as needed, to alleviate undesirable conditions that may exist. Design modifications also are tested, if feasible, in an effort to reduce construction costs and still provide highly functional harbor designs.

Design Information Required for Model Investigations

338. It is very important to determine realistic and accurate design conditions when developing a hydraulic model investigation. The designer should be sure deepwater wave characteristics (period, height, direction, spectral shape, recurrence intervals, etc.) have been well defined (measured or hindcast) offshore of the site. On the open coast, a wave refraction analysis should be conducted. When waves move into water of gradually decreasing depth, transformations take place in all wave characteristics

except wave period (to the first order of approximation). The most important are the changes in wave height and direction of travel. Wave refraction and shoaling coefficients are determined from deep water to the depth of water simulated at the wave generator in the model. The bathymetry from the wave generator to the harbor site will correctly refract and shoal the waves as they propagate over the model. Measured prototype wave data and/or hindcast data are obtained for the site to determine wave characteristics and recurrence intervals, or return periods, needed for the investigation.

339. To ensure proper wave transformation to the harbor site it is important that current and accurate bathymetric data is provided. The more detailed the data, the more accurate the model can be constructed. Shoreline details, irregularities, and overbank elevations also are important so that correct overtopping, runup, and reflective characteristics are simulated.

340. If existing breakwaters, revetments, groins, absorbers, or other structures are present, detailed, as-built data is needed on structure cross-sections, lengths, and alignments. Depending on the required model scale, adjustments are made to sizing of model construction material to ensure that correct wave transmission, overtopping, and reflection characteristics are reproduced in the model.

341. Still-water levels also are important design conditions. They are selected so that wave-induced phenomena dependent on water depths are accurately reproduced in the model. Normally, more wave energy reaches a harbor site during periods when higher water stages occur in the prototype. These may be during the higher water phase of the local tidal cycle or higher lake levels due to seasonal fluctuations. Also, most storms moving onshore are characteristically accompanied by a higher water level due to wind tide and shoreward mass transport. Conversely, however, lower water levels may result in further offshore movement of longshore sediment (i.e. around the head of a jetty into an entrance channel) since the breaker zone is further offshore with lower swl's. Prior to the conduct of a model investigation critical swl's must be defined.

342. If sediment movement alongshore is a concern at a harbor site, the design engineer should make sure that the direction of predominant sediment movement is determined prior to a model investigation. The design engineer also should determine if reversals are probable at the site. Model tests can be performed to qualitatively determine sediment movement patterns and likely

areas of accretion or shoaling. To determine the tracer material to be used, the median diameter (D_{50}) of the prototype sediment as well as its specific gravity is required. Observations or an understanding of prototype movement patterns is critical to initial validation of the model. Then, tracer material can be used in concert with incident wave climates to give qualitative indications of scour and accretion patterns and directions of predominant sediment transport.

343. In areas where harbors are constructed in river mouths and a model study is being developed, river discharge information will be required to determine the impact that any harbor modification may have on river flow characteristics. A range of river discharges for various return periods should be tested. In addition, roughness coefficients (Manning's n values) must be determined so that correct riverbed roughness and overbank roughness is simulated in the model. From these values, roughness can be applied to the model bed to simulate realistic conditions. Qualitative data on bed-load sediment movement patterns in rivers can be determined in the model. The specific gravity and median diameter (D_{50}) of river sediments must be determined prior to conduct of the model study.

344. In areas, such as inlets, where ebb and flood tidal currents in conjunction with wave energy can move sediment into a navigation opening or cause meandering of a navigation channel, tidal information will be required prior to a model investigation. Normally, prototype data are obtained over a representative tidal cycle to be used as input in the model study. Tidal elevations, tidal currents, and salinity data are generally secured during these periods. In some model investigations, the entire tidal cycle is reproduced, and in others, simulation of maximum flood and ebb tidal currents are all that are deemed necessary. Investigations reproducing the tidal cycle are difficult and time consuming to conduct but yield very valuable information concerning the site hydrodynamics. Generally, extensive model validation tests are required prior to conducting tests to determine the impacts of various improvement plans.

345. Once design conditions have been established, the model sponsor must ensure that realistic performance requirements are established against which improvement plans are judged as functionally acceptable. Performance of some of the prototype harbors in which model studies have been conducted indicates that the models accurately predicted wave conditions, but the wave

height acceptance criterion selected were excessive. This, in turn, results in many complaints by local harbor users that excessive wave conditions are occurring in the prototype. And this in turn incorrectly implies that the model did not accurately define expected wave conditions in the harbor.

346. This part of the report generally discusses model investigations and design information required for small-boat harbor studies. Although small-boat harbors may be classified in various classification schemes, each and every harbor is different. Each harbor has a different configuration as well as possible different potential problems and design conditions. The information contained herein is valid in general. For clarification on any aspect of small-boat harbor modeling, design conditions required for modeling, or site specific design problems, WES engineers and scientists should be contacted for assistance and guidance.

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Waterways Experiment Station Cataloging-in-Publication Data

Bottin, Robert R.
Physical modeling of small-boat harbors : design experience, lessons learned, and modeling guidelines / by Robert R. Bottin, Jr., Coastal Engineering Research Center ; prepared for Department of the Army, U.S. Army Corps of Engineers.
170 p. : ill. ; 28 cm. — (Technical report ; CERC-92-12)
Includes bibliographic references.
1. Harbors — Design and construction — Models. 2. Harbors — Hydrodynamics — Models. 3. Hydraulic models. I. United States. Army. Corps of Engineers. II. Coastal Engineering Research Center (U.S.) III. U.S. Army Engineer Waterways Experiment Station. IV. Title. V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; CERC-92-12.
TA7 W34m no.CERC-92-12